PERFORMANCE ASSESSMENT IN WATER SUPPLY AND DISTRIBUTION

by

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This work is dedicated to Eng.António Teixeira Coelho,
who introduced me to civil engineering.

ABSTRACT

Performance analysis is becoming a key issue in the engineering approach to the control of water supply and distribution systems, both as a natural process of evolution of the modelling and design methods available, and as a consequence of an ever increasing awareness to the quality of the service provided within the water industry today. Measuring the performance of a water system is not however a straightforward task, since it can be perceived from different viewpoints and related to a variety of parameters and properties of the network which are not always quantifiable.

This work presents a systematic approach to the analysis of performance, by creating a framework in which a variety of concepts and criteria can be included. The approach is based on the establishment of standardised performance measures, developed as an extension to the existing engineering analysis and modelling procedures. The measures are calculated from the results of conventional steady-state or extended period network analysis. It is only necessary to know the complete set of flows and heads for each modelled situation.

The set of measures identified as relevant to the performance analysis and adequate for this type of approach range from the hydraulic parameters – pressure at demand points, stability of the head surface, power usage – to physico-chemical water quality parameters and to the reliability and redundancy levels of the network. The indices translate the performance of the system relative to the particular measures by means of appropriately refined penalty curves that can be further tailored to specific requirements or the analyst's sensitivity.

The method is applied to the different areas of performance of water distribution systems and illustrated with various case studies, and its applicability to a range of engineering problems in water distribution is explored. In the process of doing so, several key areas of water networks' behaviour are analysed in detail and some advances are made in the analysis and modelling procedures that are currently available. These areas are water quality modelling, where an innovative, performance-oriented model is presented, and reliability analysis, where some existing methods based on the evaluation of network entropy are refined for the specific purpose of performance assessment.

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NOTATION

C_{0i}	Concentration of supply flow at node *i*.
C_{ij}	Concentration of flow leaving node *i* for node *j*.
C_{ir}	Incoming concentrations to a storage device from contributing nodes *i*.
C_{ro}	Outgoing concentrations from a storage device to downstream nodes *o*.
C_r	Fully-mixed concentration at storage device.
C_l^i	History of concentration values at node *i* during the hydraulic time step ΔT, to which correspond the durations t_l^i, $l=1,...,N^i$, in the discrete time sequence H^i.
C_k^{ij}	Concentration of the k'th segment in pipe connecting nodes *i* and *j*.
$\delta C_k^{j(i)}$	Concentration component of $\delta H^{j(i)}$, $k = 1,\ldots,N_e^{ij}$.
C_{HW}	Hazen-Williams' pipe roughness coefficient.
D	Pipe diameter.
D^i	Set of downstream nodes of a generic node *i*.
$\overline{\varepsilon}$	Relative error.
h	Headloss per unit length of pipe.
h_i	Pressure head at node i.
h_{min}	Minimum pressure requirement.
h_{max}	Maximum pressure requirement.
Δh	Headloss
$\Delta h_{\max}$	Maximum headloss to node.
H^i	Discrete time sequence, defined by the pair (C_l^i , t_l^i), which describes the variation of constituent concentration in the water flowing through node *i* during the hydraulic time step ΔT.
$\delta H^{j(i)}$	Contributing (nodal) sequence created in node *j* by flow originating in node *i*. Its components are ($\delta C_k^{j(i)}$, $\delta t_k^{j(i)}$), $k = 1,\ldots,N_e^{ij}$.
k	First order reaction rate constant.
k_b	First order bulk reaction rate constant.
k_f	Mass transfer coefficient between bulk flow and the pipe wall.
k_w	Reaction rate constant for the pipe wall.
L_{ij}	Length of pipe linking node *i* to node *j*.
L_k^{ij}	Length of the k'th segment in pipe connecting nodes *i* and *j*.
M	Number of links in a network
M_{IN}	Mass entering the network.
M_{OUT}	Mass leaving the network.
N	Number of nodes in a network
N^i	Number of concentration changes at node *i* during the hydraulic time step ΔT.

$\delta N^{j(i)}$	The number of elements in $\delta H^{j(i)}$
N^{ij}	Number of pipe segments of constant concentration along pipe connecting nodes i and j.
N_e^{ij}	Number of pipe segments contained in the imaginary *excess length* at the downstream end of pipe ij as a result of the application of Eqs.(5.7) to (5.13).
NL	Total number of links in a network.
NL^r	Number of links in loop r.
NLP	Total number of loops in a network.
P	Global value of the performance index.
pm_i	Value of performance index at element i (node or link)
Pw	Power necessary to supply all the network at service pressure levels.
Pw_i	Power necessary to supply node i at service pressure level.
Pw^{diss}	Total power dissipated in a network in order to supply all the demanded flows.
Pw_i^{diss}	Power dissipated in a network in order to supply the demand at node i.
q_{0n}	Supply flow introduced at node n from any external source.
q_{n0}	Demand at node n.
q_{ij}	Flow in link from node i to node j.
q_{ij}^{kl}	Proportion of flow in link ij which is destined to flow in link kl.
${}^{gh}q_{ij}$	Proportion of flow in link ij originating in link gh.
${}^{0m}q_{i0}$	Proportion of consumption at node i originating from the source supply at node m.
Q	Flow.
Q_0	Total flow through a network, equal to the sum of all supplies or the sum of all demands at any given instant.
Q_{ir}	Incoming flows to a storage device from contributing nodes i.
Q_n	Total flow through a node.
Q_{ro}	Outgoing flows from a storage device to downstream nodes o.
$RF(C_{ij})$	Reaction rate function of given parameter or substance carried in flow from i to j.
R_H	Hydraulic radius of the pipe cross-section
S	Entropy
S^i	Inflow-based network entropy.
S^o	Outflow-based network entropy.
S_0^d	Entropy associated with the distribution of demand flows in the network.
S_0^s	Entropy associated with the distribution of source flows in the network.
S_n	Entropy of node n.
S_{ij}	Entropy of flow from i to j.
S_{FJ}^n	Entropy of the flow-joining processes that occur in all the paths upstream of any particular node n.

S_n^{FS} Entropy of the flow-splitting processes that occur in all the paths downstream of any particular node n.

t Time.

t_l^i Duration of each period of constant concentration C_l^i, defined for l=1,...,N^i.

δt Water quality simulation time step.

$\delta t_k^{j(i)}$ Duration component of $\delta H^{j(i)}$, $k = 1,\ldots,N_e^{ij}$.

ΔT Time step used in an extended period hydraulic simulation.

U^i Set of upstream nodes of a generic node i.

V Velocity of flow.

V_{ij} Velocity of flow from i to j.

VOL_r Volume of water stored in storage device.

W Operator which extends performance index values across all the elements of the same type.

x_{ij} Distance along pipeline connecting nodes i and j.

β Unit conversion factor for pipe flow equation.

CHAPTER 1

INTRODUCTION

1.1. PERFORMANCE AND LEVEL OF SERVICE IN WATER SUPPLY AND DISTRIBUTION

The level of service provided by water supply and distribution systems is one of the key issues facing the water industry today. The need to cope with the increasingly competitive corporate environment and tight cost-effectiveness constraints while satisfying the current customer-oriented reference guidelines means that it is the global performance of the systems that needs to be addressed at all stages of the planning, design and operation tasks.

The main goals taken into account in the traditional approach to the design of water supply and distribution systems were the minimisation of investment or running costs subject to a simplified set of hydraulic constraints, and with a remedial response to network repairs or poor efficiency. The performance of the network was often relegated to a secondary role especially in terms of exploring how it can be affected by varying conditions throughout the life span of the system.

That tendency has been reversed in recent years, with a new emphasis being placed on the efficient analysis of the way in which the system performs its water distribution task for a variety of circumstances. These are particularly important points in the case of urban distribution networks, often characterised by complicated layouts and myriad demand points but frequently over-simplistic operation.

This type of problem is not exclusive to the design phases. It is common to find existing systems with under-design problems or functional and operational difficulties originating in hydraulic shortcomings. With the current rapid growth of many urban areas, there is a strong need for the type of tools that will allow the engineer and designer to evaluate the global performance of a system in order to facilitate diagnosis and decisions, without having to rely totally on the empirical insight of the experienced decision maker. The systematic use of network analysis models is certainly a correct path towards the solution of the problem. Water network simulators are an invaluable aid in the assessment of the system's response to alternative demand and operational scenarios. However, the type of results returned by such models can be complex and far from intuitive, frequently making their interpretation difficult and less than objective when it is necessary to compare between different situations. That is where the present work proposes to act, by providing a standardised assessment of performance focusing on a variety of aspects of water systems' operation and behaviour.

Measuring the performance and assessing the level of service provided by a water distribution network are not straightforward tasks, given the multiple factors and viewpoints involved, and the lack of a unified approach or a single clear-cut definition of performance. The concepts most commonly associated with the performance of water distribution networks have to do with the adequacy of supply in hydraulic terms, with the quality of the water provided, and with the reliability of that supply (and inherently of the network carrying it) both in quantity and quality terms. Although, in each of those specific aspects, techniques are available providing much of the relevant information, integrated methodologies that allow for flexible use in engineering tasks are not yet widespread. This work attempts to systematise the issue of performance analysis in water supply by putting together a flexible framework based on an array of measures, each devoted to a particular aspect.

The framework required must be able to define some sort of system which, for a certain domain relevant to the technical management of a water network, classifies its activity according to a scale of merit with regard to the level of service provided, to a particular notion of technical performance, or more generally to an analysis or design objective. The principal requirements for such a methodology are:

- it must be flexible enough to accommodate with ease the different sensitivities, interpretations or objectives of the analysis, given the open nature of performance assessment as discussed previously;

- it should allow for a certain degree of standardisation in order to facilitate, and indeed validate, a multi-disciplinary approach, where the various aspects to be considered may be brought down to the same quantified basis; and
- it must be quantitative and numerically based — the envisaged tool should be translatable computationally in order to afford intensive use, either from within or as post-processor to the current modelling techniques. Even though it is to a certain extent possible to deal computationally with non-numerical information, it would be desirable for the sake of simplicity to find a method which would allow for numerical treatment, especially if integration with the current analysis and modelling techniques is also a target.

These somewhat conflicting objectives are addressed in this work by means of a simple methodology, which is applied to a variety of engineering aspects of performance in water distribution, selected while reviewing the three main areas of water networks' behaviour: hydraulics, water quality and reliability.

1.2. OBJECTIVES OF THE PRESENT WORK

The main objectives of the present study can be summarised as follows:

i) To analyse the concept of technical performance in water supply and distribution and identify areas of study which may lend themselves to an engineering approach.

ii) To develop a systematic and quantifiable approach to performance evaluation that may be used as a common methodology when tackling different areas of performance analysis. The method should also be designed as an engineering tool to complement the existing modelling and analysis techniques.

iii) Finally, to analyse each selected area of study in some detail; to identify what aspects may be suitable for the approach mentioned in ii), and how to model and quantify them; to apply the performance evaluation methodology and analyse system performance based on those aspects.

1.3. METHODOLOGY AND LAYOUT OF THE PRESENT WORK

The present text is organised in five main chapters. After an introduction to the subject of performance in water distribution, Chapter 2 reviews the different sides of the problem, from the engineering point of view to the regulatory levels of service framework and the consumer's perception. The main areas of study for the present work are identified and outlined.

Chapter 3 develops and discusses a multi-purpose framework, based on quantitative measures of performance assessment and suitable for use in the operational and technical management environment of water supply and distribution utilities, as systematically and automatically as possible, and as a complement to the existing modelling and analysis capabilities. The method devised is based on penalty curves, applied to the values of those network properties or state variables which are perceived as appropriately representing the aspect or aspects being analysed. A standardisation procedure of performance is established in such a way that a fixed range of values is used to define the various levels, from optimum service to no service. That classification is then applied at elementary level and subsequently generalised across the network by means of an appropriate operator. The information thus obtained is organised in graphical form both for extended period simulations and, when applicable, for a range of demand loads.

The various components of the method are described, with a reference to the assumptions and simplifications which are inherited from the main supporting tool, network analysis. The requirements and desirable properties for the measures contemplated by the scheme are discussed, as are the various ways of presenting the results. The method is implemented through a computer program, PERF, which serves as the framework and main program for a series of domain-specific programs for performance assessment, developed in Chapters 4 to 6.

Chapter 4 reviews what is inevitably the first area to be explored in an evaluation of a water distribution system's performance: its hydraulic behaviour. This chapter applies the previously introduced performance assessment framework to such hydraulic characteristics of water distribution systems as pressure head and flow velocity. Since the whole system to be developed is based on state variables whose values are to be obtained through modelling of the network, it is important to highlight the main characteristics, uses and limitations of the existing network analysis methodologies. After introducing the subject of hydraulic modelling in supply and distribution networks, the text reviews the main types of models, describes the general formulation governing the processes to be modelled and highlights the solving methods.

The selection of what hydraulic state variables to include in a performance evaluation system is then presented and measures concerning pressure, pressure fluctuation, flow velocity and energy consumption are proposed. The corresponding penalty curves and generalising functions are discussed, as well as the suitability of the various measures to the framework proposed and to the objectives of the work. Illustrative examples are given and the use and potential of the methodology explored.

The second performance area explored in this work is concerned with the quality of water distributed and is described in Chapter 5. As with the previously explored performance area, water supply and distribution companies are required to meet service standards relating to the potability and aesthetic aspects of the water delivered to their customers. Potable water must meet restrictions on its microbiological contents, as well as on the concentration values of chemical, biochemical and physical substances carried with it. Water quality will vary in space and time across the network, often with deterioration of its aesthetic properties — odour, taste, colour, turbidity — and of its chemical, physical and microbiological contents, bringing about the danger of contamination.

Chapter 5 proposes to apply the standardised performance assessment framework to the field of chemical, biochemical and physical water quality in distribution networks. The first step is to select relevant variables and obtain their values through appropriate modelling. In contrast to the hydraulic performance measures, for which there are well known and widely available models, water quality modelling is a less developed domain. Not only are the commercial or public-domain models less available than their hydraulic counterparts, but the techniques documented for those or published in the literature still lend themselves in most cases to some improvement. The present work therefore includes the complete design of a water quality model. Its development and implementation is presented in detail, after a detailed review of the existing methodologies. A complete description is made of the numerical algorithm for solving the dynamic water advection and mixing formulation, both in flow through the pipeline system and through storage and other devices. The suitability of the algorithm is discussed with emphasis on numerical accuracy, and different solutions are presented for a numerical diffusion problem arising from computing limitations. The model is extended in order to carry out not only the modelling of constituent concentrations but also the calculation of travel time and source contribution . A computer implementation is presented, and application examples discussed in order to highlight the different aspects of the model. The development of penalty functions and the corresponding generalising functions for some of the most typical water quality problems faced by distribution system managers are then discussed with the help of illustrative examples. The suitability of the performance framework for water quality is analysed and concluded upon.

Chapters 4 and 5 analyse some of the most palpable aspects, both for the designer or technical manager and for the consumer, of the performance of a water distribution system. Generally speaking, those aspects translate the very objective of a water utility: to satisfy all demands with sufficient and wholesome water, at adequate pressure, and at the minimum possible cost. Further to finding out to what degree that objective is accomplished, the performance of a water supply and distribution system can also be measured by how consistently or *reliably* it actually does so. Reliability of water distribution networks is the subject of Chapter 6. In contrast with the previous two chapters, where it was relatively straightforward to identify at least some relevant performance aspects and the properties or state variables that translated them, being more the case of reaching a satisfactory or efficient method of their calculation, here it is less clear what exactly the analysis is attempting to measure, let alone finding the property or variable that translates it. The most important methods for reliability evaluation described in the literature are analysed in order to tackle that problem. The review divides the available techniques into direct and indirect methods, and discusses how the concepts of reliability and redundancy can be associated and how the latter may be better evaluated using indirect techniques. The use of maximum entropy flows is one of the main methods for indirect or surrogate evaluation of reliability, and is selected as the basis for the reliability performance evaluation proposed in this work. The most relevant points of the entropy maximisation methodology are introduced. A new formulation is then proposed which corrects or completes some of the published methods, followed by a discussion on the suitability of entropy maximisation for reliability evaluation. The last section of this chapter applies the performance evaluation framework to the reliability measure, discussing the possible uses, corresponding penalty curves and generalising functions, and illustrating with some examples.

Finally, Chapter 7 presents the main conclusions of this research and suggests some areas for further research.

CHAPTER 2

PERFORMANCE IN WATER DISTRIBUTION

"Performance n., the fulfilment of a claim, promise or request" [1]; *"the manner in which or the efficiency with which something reacts or fulfils its intended purpose"* [2]

2.1. INTRODUCTION

The ability of an existing or planned water supply and distribution system to perform adequately – that is, to fulfil appropriately its intended purpose – under the widest possible range of likely operating conditions, particularly those that are expected to occur during its working life, is a crucial system characteristic. The likely performance of a distribution system is not often assessed in its relevant globality or even explicitly defined in water supply engineering, which traditionally approaches its tasks from a relatively fragmented perspective and has difficulty in formulating its methods for the complete range of operating conditions that are in reality met by the systems.

Water supply and distribution networks are designed, built and run in order to fulfil an apparently simple objective: to supply people with water. However, they frequently form such complex systems, in conjunction with the way they are operated, that the diversity of problems raised by their management quickly overwhelms that apparent simplicity. Many different objectives are pursued by the various types of analyses, procedures and policies developed to support the planning, design and operation of water distribution systems. Traditional engineering design is based on minimisation of cost factors provided some simplistic hydraulic constraints are respected. Optimal operation will look at pumping or disinfection efficiency, again subject to some simple constraints of a hydraulic nature. Leakage control concentrates mostly on excessive pressure reduction, without much concern for the remaining performance of the system, and so on. This diversity of objectives makes it difficult

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1 (Collins English Language Dictionary, 7th Ed.)
2 (The Random House Dictionary of the English Language, 2nd Ed.)

at any moment for the water engineer to address the overall performance of a water supply and distribution system in a balanced manner. However, the tendency in the water industry itself, driven by a market-oriented need to provide its customers with the best level of service at the lowest possible cost while satisfying the regulatory framework, is to progressively take into consideration and reduce to the same basis all the different aspects of water distribution that may be subject to informed or not-so-informed scrutiny.

The present chapter briefly reviews the main points of view and priorities in modern day water supply, while attempting to identify the topics which are most relevant for the development of a performance assessment methodology, as a complement to the existing engineering analysis and modelling tools.

2.2. THE CONCEPTS OF PERFORMANCE AND LEVEL OF SERVICE IN WATER DISTRIBUTION

2.2.1. The water industry regulatory framework and the consumer's perspective

The current trend in most services towards a greater requirement for quality and consumer satisfaction is particularly felt in such essential infrastructures as water supply utilities. In England and Wales, the Water Act 1989 and the Water Industry Act 1991 established a regulatory framework which is based on the definition of quantifiable levels of service, as targets to be met in a variety of aspects relevant to water supply and distribution. The levels of service scheme is not only an instrument of control of the water companies' activity, utilised and enforced by the competent government body, the Office of Water Services (OfWat), but it also constitutes a reference setting for the same water companies, around which their own strategies are planned. As regards drinking water, the levels of service contemplate mainly hydraulics and continuity of supply. Water quality in the distribution networks is verified by the Drinking Water Inspectorate, subject to a different and specific set of national regulations and international directives which have from the outset been established in a much more quantified and demanding basis than their hydraulic counterparts, due mainly to the essential public health implications of their enforcement.

One of the most relevant aspects of the modern water industry environment is the predominant role of the consumer, progressively the focal point of the whole process. Companies as well as regulating bodies are increasingly aware of (and conditioned by) consumer protection issues and exposure to public opinion. The current trend in many countries towards some form of private management or even ownership of infrastructure services has made that effect even more noticeable.

In line with other recent regulations and legislation that increasingly pay greater attention to consumer protection, the 1989 Water Act establishes clearly that the consumers must be granted access to publications where their rights are presented in a clear and accessible way; that they have the right to freely examine water supply records at any time; and that water supply and distribution companies must publish their levels of service performance results, that is, how they fared over a given period (normally, on a yearly basis) as regards the minimum levels of service guidelines legally established. Guaranteed Standards Schemes have been set up to ensure the consumer a direct monetary compensation of a pre-specified amount, by day or by event, in case of infringement of the minimum standards by the water utility.

Water companies are consequently under the obligation to present annually to the Director of Water Services an activity report regarding their performance during the previous year. That report is specifically written in terms of a set of level-of-service indicators defined by OfWat for water supply (as well as wastewater), which compare the actual service delivered to the customers with given reference levels. The DG[3] level-of-service indicators contemplate, among other aspects: the availability of water in bulk (percentage of population whose bulk demand has not been met, as compared to a reference level); pressure in the distribution network (number of consumers at risk of being supplied at pressure levels lower than the reference level); interruptions to supply (number of consumers affected by interruptions to supply lasting longer than the reference duration, without

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[3] DG is an acronym for Directorate-General (of Water Services)

advance warning and appropriate justification from the water undertaker; and restrictions to water use (such as number of consumers subject to hose pipe bans, etc..).

The standards have been developed for the most important aspects of the service delivered, considering the objectivity of their evaluation and the possibility of direct quantification of results. For each standard, a criterion is defined which grades the service as acceptable or unacceptable. As an example, the standard referring to interruptions to supply defines that the level of service is unacceptable when the supply at the consumer's stop tap is interrupted for more than 12 hours, be it due to system design inadequacies, water shortage at source or network element failure. Interruptions due to maintenance or repair works with advance warning are excepted, as are those caused by third parties (from the point of view of the water undertaker), such as power failures or accidental damage to the system. The reports must mention all such interruptions lasting more than 12 hours, as well as those between 8 and 12 hours, which are considered significant even if not violating the standard. The time, duration, number of affected customers, cause and remedial action must be reported. Similarly thorough rules have been established regarding the other standards. The final, synthesis report must include not just the total compliance figures but also statistical bands of confidence calculated according to precise guidelines.

2.2.2. Level of service from the water supplier's point of view

Water supply companies have in turn created their own level of service verification procedures. These are as much a means of guaranteeing compliance with the legal requirements – and in this respect are often even more stringent for reasons of self-protection – as guidelines that condition corporate strategy and internal management policies.

The basic acceptable.vs.unacceptable classification is, at the internal level of the water company, frequently replaced by a more graded perspective of performance. The basic requirement is often raised (for example, most companies keep an internal minimum pressure standard which is 50% to 100% higher than that imposed by the relevant DG), but then several types of levels of service are defined, from the design level of service (the target level of service to be achieved by a new network or expansion) to trigger levels which determine the introduction of corrective action, within the strategic planning of the company, to the minimum levels below which the company is not delivering and must compensate its customers, undergo OfWat imposed corrective action, or even face legal proceedings. These latter minimum levels are normally made to coincide with the DG standards.

Since it is impossible to monitor continuously all points of consumption, most companies resort to the use of network analysis and simulation, combined with localised surveys, district metering results and telemetry, to complement the customer complaint records in the evaluation of the performance of their systems. The areas of greater concern for the water undertakers are those where the consumer is particularly sensitive: continuity of supply, followed by pressure and *perceptible* quality problems.

The importance of correct use of network analysis and simulation in this context is very high, as it provides the best but also often the only means of finding out what is happening throughout the network. Some companies also rely on computerised information systems that register and process consumer complaints. In this respect, a sophisticated solution such as the Water-SIR system developed by Severn-Trent Water (Lackington, 1991), is particularly relevant as it records the consumers' perceptions of such level-of-service related variables as pressure, taste, odour and colour. It has become important to supply the customer with a product which is not only proper and safe, but also perceived as such.

For the water company, the level of service is thus understood as not only the compliance with the accepted industry practice and the regulatory framework, implicitly incorporating all the identified quality requirements for proper drinking water, but also, and quite importantly, the provision of a *pleasant* product. In other words, something that the customer can see and perceive as good quality, hence good service, thus improving *de facto* service levels and the image of the company. References can frequently be found (e.g., Tansley and Brammer, 1993) to the fact that the consumer is normally willing to pay extra for a quality-oriented product.

2.2.3. Performance concepts in water supply engineering

It has been mentioned that the subject of performance of a water supply and distribution system has at least as many meanings for the water engineer as the diversity of analysis procedures commonly employed in the planning, design and operation of the systems. It is commonly understood that the system should be designed to accommodate a set of demand points, providing the necessary flows at the required pressures. A layout is established given the terrain and spatial constraints, and the least-cost solution that would correspond to a tree-type configuration usually discarded in favour of the inclusion of some strategically placed loops that guarantee alternative paths of supply, not just to ordinary consumption but also to fire-fighting flows and other crucial demands. The pipes, reservoirs and pumps are sized for minimum investment and operating costs, while pumping schedules are adjusted to reduce storage space or energy costs. Disinfection and other water treatment procedures are subsequently developed to accommodate the public health guidelines and regulations.

While all such aspects have their own degree of importance, the different objectives that those procedures pursue are not always seen in an integrated, global perspective, or the performance of the system clearly defined as such. In fact, the levels to which modelling and simulation techniques have been developed in water supply engineering have not been corresponded so far by much specific work in this particular area. Whenever the subject is pinpointed, most authors seem to associate the idea of performance of a water network mainly to its reliability characteristics (e.g., Hashimoto *et al.*, 1982; Mays, 1993; Tanyimboh, 1993). Hashimoto *et al.* (1982) base the assessment of performance on system failure, defined as any output value in violation of a performance threshold, such as a regulatory standard or contractual obligation. System performance is defined from three different viewpoints: i) *Reliability*, or how often the system fails; ii) *resiliency*, or how quickly the system recovers from failure; iii) *vulnerability*, or how serious the consequences of the failure may be.

La Loggia and Mazzolla (1989) describe an attempt to test the efficiency of water allocation alternatives by measuring the performance of the overall system through a set of indices. These are based around the concepts of vulnerability (in the sense referred to by Hashimoto *et al.*), with 3 performance indices computed over a period of years for annual average, monthly average and overall maximum values of the intensity of shortage events, measured by the percentage shortfall; 2 other indices are reportedly related to the system reliability and define respectively the percentage satisfaction of target demands, and the percentage of time the system is fully meeting target demands; and 3 further indices measure other time-related statistics, such as mean time between failure, average and maximum failure event duration. These indices are then weighted together to provide overall measures of the performance of the network for different combinations of targets.

This type of approach, as most based on reliability considerations, is geared towards an assessment of past performance of the systems, if and when a time history has been recorded. Apart from the fact that such records are not often available, this does not solve the problem typically faced by water engineers of analysing systems that have not been built yet, future extensions, alternative operating scenarios, rehabilitation options, etc.. Generally speaking, the widely used network analysis and design models, on which much of the engineering activity carried out within the water industry is based, have been developed over the years with a fragmented view of the aims pursued and without explicit consideration of performance issues in the way that the water utilities are increasingly forced to acknowledge. In some ways, engineering practise has been slow to catch up with the modern regulatory environment (as discussed in the previous section) and the driving forces in the water industry. It is to this area of work that the present research effort aims to contribute.

2.3. MEASURING PERFORMANCE IN WATER DISTRIBUTION

The very fact that the goals pursued by water utilities have been somehow put in a new and more stringent context by the modern regulatory frameworks means that the engineering tools used in the support of their activity should reflect those needs, and be able to reformulate the tried and tested procedures in this new light. As seen before, it is not always practical for the network analyst and water engineer to find out about the performance of the network *a posteriori*. Industry levels of service are usually assessed based on a run of recorded events over a period of time, which permits the build up of statistical data. These are of great value when available, not only for supporting direct intervention studies but also as a basis for extrapolation. However, many engineering procedures employed in direct support of design, analysis and control tasks of a water network need to be able to simulate a great variety of hypothetical situations and scenarios. Different alternatives have usually to be tried as solutions to problems whose direct, mathematical optimisation is often complicated by a multitude of parameters and a difficult understanding of the systems' behaviour. This means developing procedures to analyse and measure water networks' performance on an *a priori* basis, that may fit the type of analysis carried out by network simulation and be used for the same purposes.

As Hashimoto *et al.* (1982) emphasise, it would be useful to capture in a standardised manner particular aspects of water network performance that may be of importance for the generality of tasks pertinent to planning, designing and operating those systems. It would be particularly interesting to be able to analyse wide ranges of operating conditions under the same basic approach, and with the possibility of simultaneously finding out about several different aspects of the systems' behaviour to increase awareness and sensitivity to the less obvious aspects or characteristics. Overall, such an approach should prove rather useful in the decision-making processes that drive the selection of system layouts, capacities, operating policies and ideal configurations.

The first step in that direction is a broad selection of the main areas of water systems' performance that may be prime candidates for detailed dissection and application of such an approach. Those main areas have already been introduced in the opening chapter of this work. The first and most obvious domain concerns the hydraulic behaviour of the network. The processes of conceiving, designing, building and running a water supply system are primarily driven by the need to satisfy a given set of demand points with sufficient flow of water at usable pressure levels. That has always been not only the prime motivation of engineers and designers, but is also central to the regulatory environment of most countries. Measuring the hydraulic performance of a network is therefore crucial in any attempt to develop a system such as mentioned above.

Water supply and distribution companies are naturally also required to deliver wholesome water and must meet service standards relating to the potability and aesthetic aspects, not just for the sake of their customers' health, but also as a matter of acceptability. Potable water must meet restrictions on its microbiological contents, as well as on the concentration values of chemical, biochemical and physical substances carried with it, as its quality will vary and often deteriorate in space and time across the network. Water quality is therefore the second major area of concern as the accomplishment of a system is tested. This, however, is an area that only began deserving due attention from engineering practice much more recently than the hydraulic domain. Many of the existing methodologies have been developed on top of, sometimes almost grafted on to, the modelling and design procedures previously evolved for the hydraulic analysis, and often suffer from an after-thought effect. They are still particularly overlooked in the design stages and function very much as secondary verifications. It is therefore important to redress this conceptual vice by approaching water quality performance simultaneously and, as much as possible, under the same light as the hydraulic performance, in order to try and obtain a better balance of priorities in the analysis and simulation procedures.

Other aspects of the behaviour of water systems may be isolated for performance analysis, but are at present less relevant in terms of what has been said in the previous section. It will be seen throughout the present work that the two major areas identified above provide a good first approach to the selection of evaluation criteria, and can in turn give rise to an interesting range of aspects for performance assessment.

Finally and for obvious reasons, both the water companies and the regulatory bodies need to find out (or be able to demand) the level of reliability with which the systems perform to the levels of service which are established in operational – hydraulic, water quality and other – domains. It has been mentioned before that reliability of the systems is an area where some specific performance evaluation proposals have been made in the literature. It is therefore important to include in an analysis of system performance its reliability characteristics.

2.4. SUMMARY AND CONCLUSIONS

This chapter briefly reviews the concepts of performance and level of service in water supply and distribution, in order to place in adequate context the present work. Performance is analysed from the different viewpoints which prevail in modern day water distribution, namely those of the water distribution companies, the regulators and the consumers. The various aspects that must be taken into consideration by these parties and the customer-satisfaction oriented framework that is increasingly taking over the priorities of both regulators and water utilities are ventilated in an attempt to isolate the most appropriate way to address the subject.

Performance evaluation in water supply engineering, the central theme of this work, is reviewed in the light of the modern trends in the water industry. It is concluded that the diversity of objectives pursued in the various procedures and methods traditionally employed in the technical support to planning, designing and operating water supply and distribution systems makes it difficult at any moment for the water engineer to address the overall performance in a balanced manner. The need is identified for a standardised and systematic approach to the assessment of particular aspects of water network performance that may be of importance in the decision-making processes that drive the selection of system layouts, capacities, operating policies and ideal configurations. A broad selection of the most relevant areas of study for that purpose has isolated the hydraulic, water quality and reliability fields as the main subjects to analyse.

The present study attempts to contribute to this technical domain by developing a systematic approach to the evaluation of performance in water supply and distribution, based on a quantified appraisal of a water distribution network's behaviour as compared to pre-specified objectives it sets out to accomplish. The next chapter introduces the basic framework created for that analysis.

CHAPTER 3

A PERFORMANCE ASSESSMENT FRAMEWORK

3.1. INTRODUCTION

One of the objectives of the present work is to develop a systematic approach to the evaluation of performance in water supply and distribution. As argued earlier, this should be based on a quantified appraisal of a water distribution network's behaviour as compared to the objectives it sets out to accomplish. The previous chapters have introduced the subject and presented the various types of objectives and related domains which may be of relevance to the system's performance, as seen from the technical management and operational viewpoints. The following chapters attempt to dissect the most relevant of those domains in some detail. For that to be carried out in a structured and meaningful manner, it would be desirable to begin by defining a unified and systematic approach that may be applied to the various fields and facilitate the integration of the different concepts explored.

This chapter develops and discusses a multi-purpose framework, based on quantitative measures of performance assessment and suitable for use in the operational and technical management environment of water supply and distribution utilities, as systematically and automatically as possible, and as a complement to the existing modelling and analysis capabilities.

The framework required must allow for the definition of some sort of system which, for a certain domain relevant to the technical management of a water network, classifies its activity according to a scale of merit with regard to the level of service provided, to a particular notion of technical performance, or more generally to an analysis or design objective. The principal requirements for such a methodology are:

- it must be flexible enough to accommodate with ease the different sensitivities, interpretations or objectives of the analysis, given the open nature of performance assessment as discussed previously;

- it should allow for a certain degree of standardisation in order to facilitate, and indeed validate, a multi-disciplinary approach, where the various aspects to be considered may be brought down to the same quantified basis; and

- it must be quantitative and numerically based — the envisaged tool should be translatable computationally in order to afford intensive use, either from within or as a post-processor to the current modelling techniques. Even though it is possible to a certain extent to deal computationally with non-numerical information, it would be desirable for the sake of simplicity to find a method which would allow for numerical treatment, especially if integration with the current analysis and modelling techniques is also a target;

These somewhat conflicting objectives are addressed in this chapter by means of the methodology presented in the next section and utilised subsequently throughout this work.

The method devised is based on penalty curves, applied to the values of those network properties or state variables which are perceived as appropriately representing the aspect or aspects being analysed. A standardisation procedure of performance is established in such a way that a fixed range of values is used to define the various levels, from optimum service to no service. That classification is then applied at an elementary level and subsequently generalised across the network by means of an appropriate operator. The information thus obtained is organised in graphical form both for extended period simulations and, when applicable, for a range of demand loads.

The various components of the method are next described, with a reference to the assumptions and simplifications which are inherited from the main supporting tool, network analysis. The requirements and desirable properties for the measures contemplated by the scheme are discussed, as are the various ways of presenting the results.

The method is implemented through a computer program, PERF, which serves as the framework and main program for a series of domain-specific programs for performance assessment, developed and used for Chapters 4 to 6. A description of PERF is included in the present Chapter.

3.2. A GENERAL FRAMEWORK FOR PERFORMANCE ASSESSMENT IN WATER DISTRIBUTION

3.2.1. Introduction

The system for performance assessment in water distribution presented in this work is centred around indices based on the analysis, from specific viewpoints, of the network's characteristics or behaviour. The method is defined by three types of entities:

- the numerical value of a network property or state variable, which is deemed to be expressive of the particular aspect being scrutinised;

- a penalty curve which maps the values of that variable onto a scale of performance for each network element; and

- an operator which allows the performance values to be aggregated across the network or parts of it.

This method yields values of performance evaluation for every element of a network as well as for the network as a whole. On the one hand, therefore, there is a *global* value which is achieved through a particular operator in order to represent the performance assessment of the network, and conversely, a population of elementary values which lends itself to a basic statistical treatment. The two are combined together graphically in diagrams where performance is plotted against a series of operational conditions, typically either a 24-hour simulation or a range of demand loads.

These aspects are detailed in the following sub-sections.

3.2.2. Relevant network property or state variable

Having determined the domain in which performance is to be measured — say water quality, as dealt with in Chapter 5 — and within it, the particular aspect of interest — for example, disinfection levels — there now must be found a network property or state variable which will best translate it.

The network characteristics and topology will normally be known. The tool of choice for providing the values of state variables, for any given state or scenario of interest, is network analysis and all its associated models, including water quality models. It must be noted though that the methodology is valid whatever the source of the values for the state variables. Any other estimates or indeed direct measurements are naturally usable if available in the suitable format. This tends however to be rare, except in a limited sense as provided by telemetry.

It has been mentioned before that the present methodology aims to be very much complementary to classical network analysis and simulation as a technical support tool for operational management of water supply and distribution systems. In fact, as will be seen repeatedly throughout this work, the best *positioning* for it is ultimately within a network analysis model.

In practice, then, network analysis will be the primary and most appropriate source of data. The point that should be emphasised at this stage is that the assumptions and simplifications of network analysis are by necessity inherited by the present method. Particularly, that a network is described in a mathematical simulation model by a set of *nodes* — which essentially represent pipeline junctions, changes in pipe size or pipe roughness, water intakes, consumption points, etc. — connected by *links* — mostly pipes but also pumps, valves and any other pieces of equipment that may be found joining two conventional nodes.

The adoption of this modelling convention means that the choice of network property or state variable to measure the performance in the field concerned must be made at the *network element* level, that is, either at the node or at the link. Taking the disinfection example which was mentioned earlier, chlorine residual concentration would probably be the appropriate variable, and would be defined at the node.

One further aspect to note is that the accuracy of the methodology cannot be greater than that of the original information about the network's state variables. Whatever the source of the data, the method is essentially based on a performance-oriented interpretation of it. It can hardly compensate for poorly calibrated models or other sources of inaccuracy, even though it can be used as an aid to gaining sensitivity to the origins of such errors.

3.2.3. Penalty curve

The second basic element of the performance evaluation methodology is a penalty curve, which plots the values of the performance index credited to the state variable or network property, at network element level, over a given range. The performance index varies between a no-service and an optimum-service situation, and the curves are supposed to penalise any deviation from the latter.

Penalty curves are arbitrary by nature and are intended to translate a given sensitivity to the relationship between the behaviour of the variable in question and some notion of system performance. The basic idea is primarily related to the concept of level of service, hence the "no-service to optimum service" classification, with the curves interpreting a common-sense, standardised grading of performance.

In actual fact, the curves can be made to mean almost anything that the modeller may have in mind for a specific analysis or diagnosis procedure — in essence, how the decision variable is rated for a given purpose over an operative range. This is where the methodology's flexibility

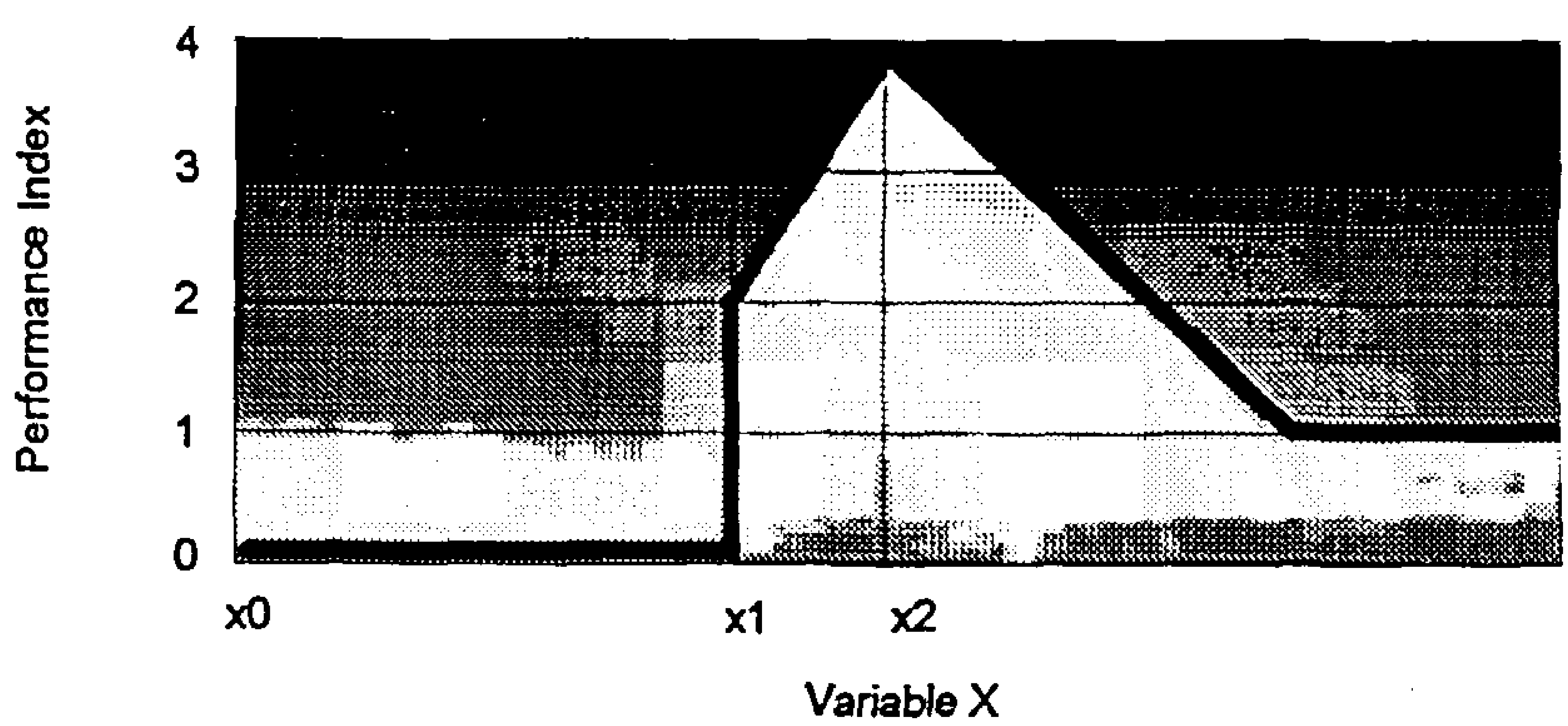

Fig.3.1 - Penalty curve

resides. However, it is worth noting that the curves should be kept relatively simple: sophisticated penalty curves may produce results that are less easy to interpret, as will become clear in the following chapters.

The convention adopted in this work establishes a scale from 0 to 4, with the following meanings: 4 - optimum service, 3 - adequate service, 2 - acceptable service, 1 - unacceptable service and 0 - no service. A penalty curve example is given in figure 3.1.

3.2.4. Generalising function

Having obtained a performance index for each network element of the type in question — node or link — it is now desirable to calculate a global value for the system.

The term *generalising function* is used in this work to designate the operator that extends the element-level performance rating across the network, producing zonal or network-wide values. The indices are intended to have both local and network-wide meaning. The generalising function is of the following form:

$$P = W(pm_i) \tag{3.1}$$

where P is the global value of the performance index, pm_i the value of the index at element i (node or link) and W is an operator which extends across all the elements of the same type. As an example, W might simply be an average across the network:

$$P = \frac{1}{n}\sum_{i=1}^{n} pm_i \tag{3.2}$$

Other types of operator may be used, such as weighed averages or those which focus on maximum or minimum values. In fact, the type of operator depends on the objective of the analysis. For most water quality parameters, the regulatory approach would look for the single worst case across the network, in which case the generalising operator would be the 0% or 100% percentile (i.e. the minimum or maximum value). However, for the same parameters, a designer analysing different alternatives might be more interested in other percentiles or weighed averages as a general approach.

3.2.5. System curves and extended-period simulation curves

The presentation of performance measures is an important feature in their usefulness for operational and appraisal purposes. Graphical representation is the ideal vehicle to convey the type of information provided by the indices and to best discern and develop any possible useful combinations. The logical step at this stage would therefore be to produce some standardised graphs of the indices' variation over different domains, that could serve as a basis for analysis and comparison.

The factors that can most influence the hydraulic performance of a system and make it vary with time are the system's physical characteristics, operational conditions and the demand loading. Of the three, demand loading has the most significant variability over time, and it is probably the only factor that lends itself to a systematic and general analysis of its influence on performance. In other words, it can be made to vary continuously over a given range with precise and meaningful significance. On the other hand, a combination of those factors can always be incorporated in a real-life type situation, such as an extended-period simulation.

The two types of graphs chosen here and standardised throughout this work correspond therefore to two simulation exercises. The first, named a *system simulation*, and the corresponding *system graph,* consists of a simulation based on an average, off-peak, demand situation and explores the potential behaviour of the system over a hypothetical range of demand loads. After selection of an appropriate average demand scenario, for which the load factor is 1, demand is made to vary over a certain range and the system is simulated statically for each step of that range [1]. The system's performance index is calculated at each time, and is then plotted against the demand range to give the system graph. In order to standardise results, it is useful to divide demand values by the corresponding current daily average and actually refer only to load factors. This makes the graph dimensionless and independent of network type and size. Figure 3.2 shows a system graph.

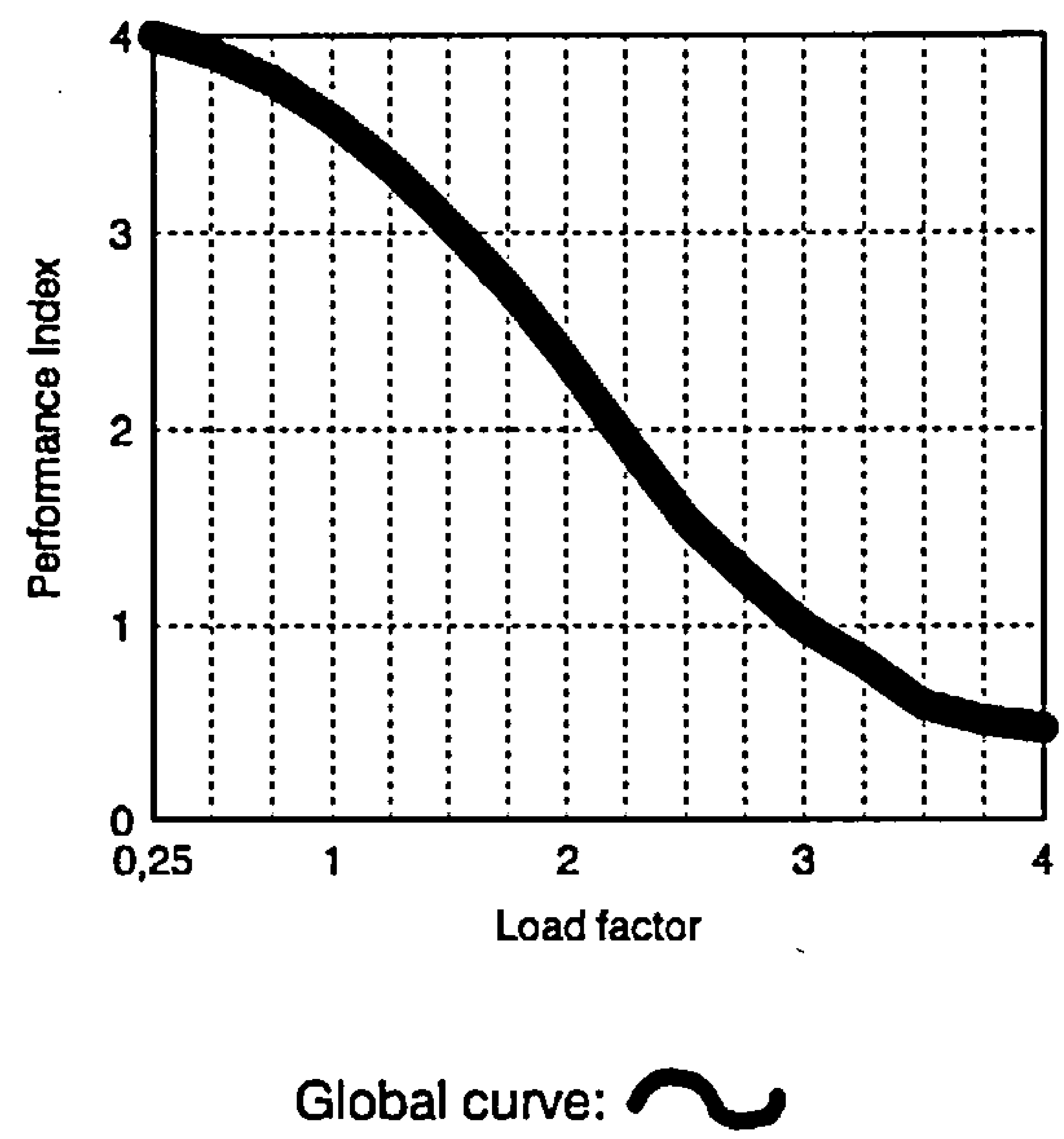

Fig.3.2 - A system graph

The range of demand loads over which the analysis is to be performed is the first decision to be taken. If the analysis is made in terms of the daily operation of the system, then the domain to be examined will range from minimum night flow to diurnal peak, or a given amount beyond that in order to test the response to exceptional demand situations. If conversely it is necessary to look into medium or long-term periods, the range must be extended accordingly. In practice it is found that a range of between 0.25 and 4 times the average demand, in 0.25 steps, as shown in figure 3.2, is adequate for most purposes.

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1 In most dynamic network analysis packages, it is possible to actually automate this procedure by simulating over a period of time for which the demand profiles have been defined as the given demand range. Precautions must be taken to ensure that the simulation is effectively a sequence of independent static

The system curves are not always applicable in performance assessment. There are situations where the variability of operational conditions, regardless of the foreseeable demand factor, is the prevailing factor. There are also certain domains where performance is time-dependent — such as those properties that depend on the travel time of the water — and for which it makes more sense to simulate and analyse a time span of operational activity.

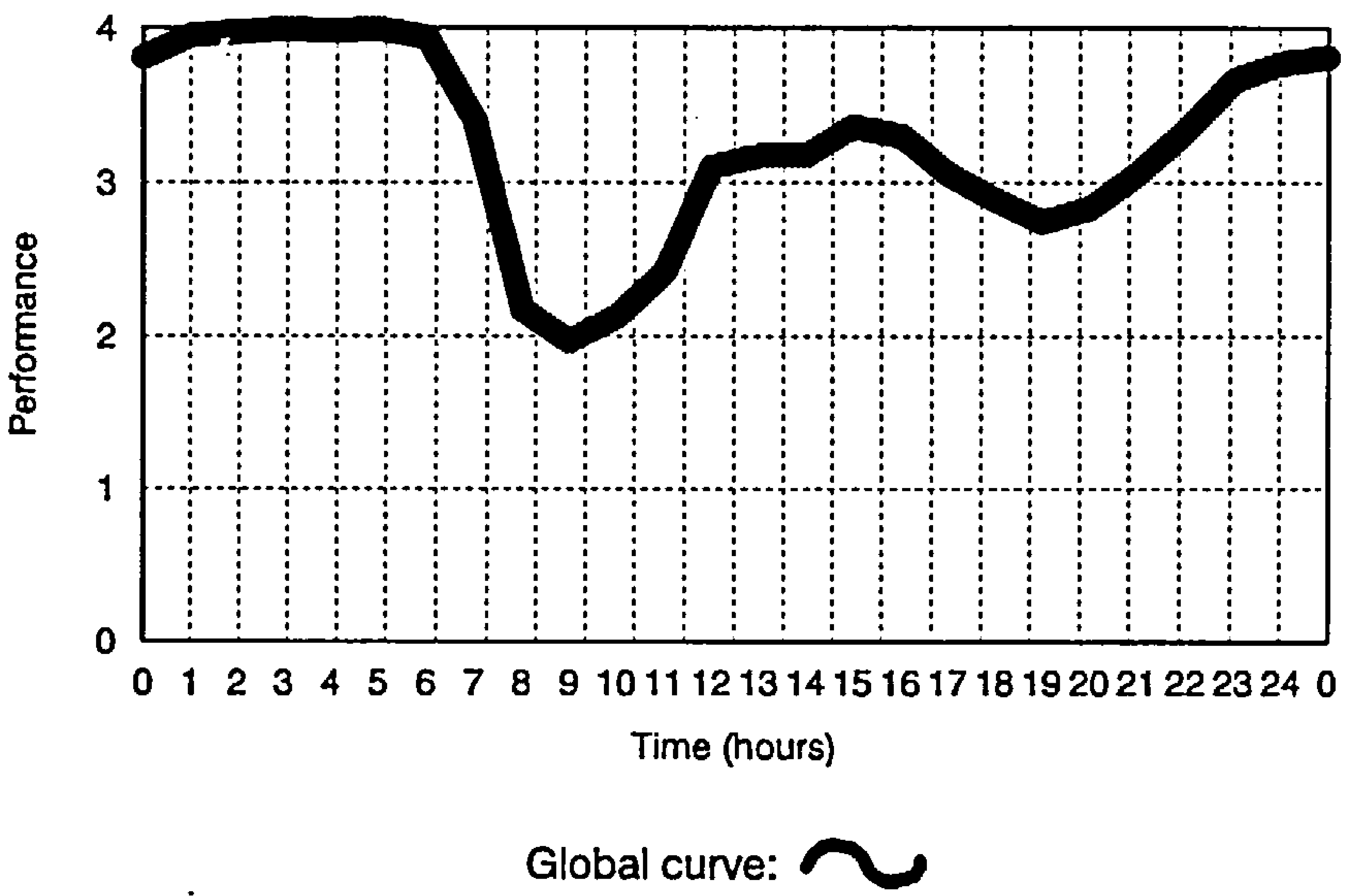

Fig.3.3 - An extended period graph

The second type of graph is therefore an *extended-period graph* and plots the performance index of the system during a dynamic simulation over a period of time, typically 24 hours. This is a classical simulation where the required combinations of operational conditions are tested over a standard time-based variation. Figure 3.3 shows an extended-period graph.

simulations, such as modelling reservoirs as infinite capacity (so that level variations do not take place) and preventing other such dynamic effects.

3.2.6. Variation Limits

However well chosen may the generalising function be, and however representative the *global* index, it cannot tell much about the spread of values that generated it. It would therefore be rather desirable to select some compact means of representing that type of information, in the system and extended simulation graphs, and avoid the time-consuming examination of the elementary data.

Variation in performance can be depicted in a number of different ways, a simple choice being to plot the extreme curves, that is, the absolute lower and upper boundaries of the index values. However, this may not be good enough, since when the boundaries are too far apart it is hard to tell whether such variation corresponds to a few insignificant outliers or to entire network areas with distinct behaviours.

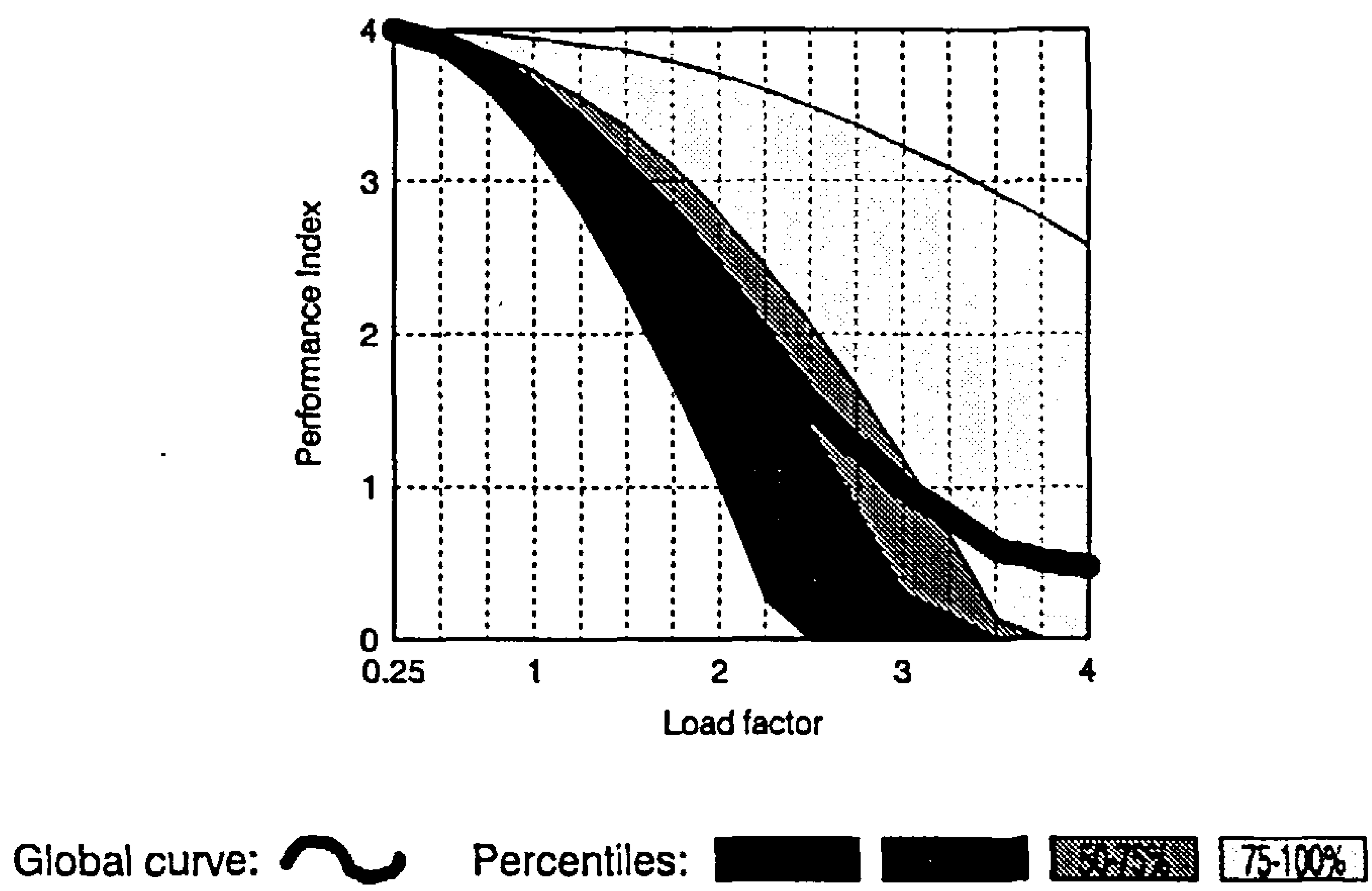

Fig.3.4 - A system graph with variation limits

It is convenient therefore to define intermediate curves, preferably with a direct physical meaning. This is provided by conveniently spaced percentiles. Figure 3.4 shows a system graph with four 25% percentile bands. These should be read as follows: if *(x,y)* are the co-ordinates of a given point in the *P%* percentile curve, it means that for a load factor of *x*, the percentage of water delivered with a performance index smaller or equal to *y* is *P%*.

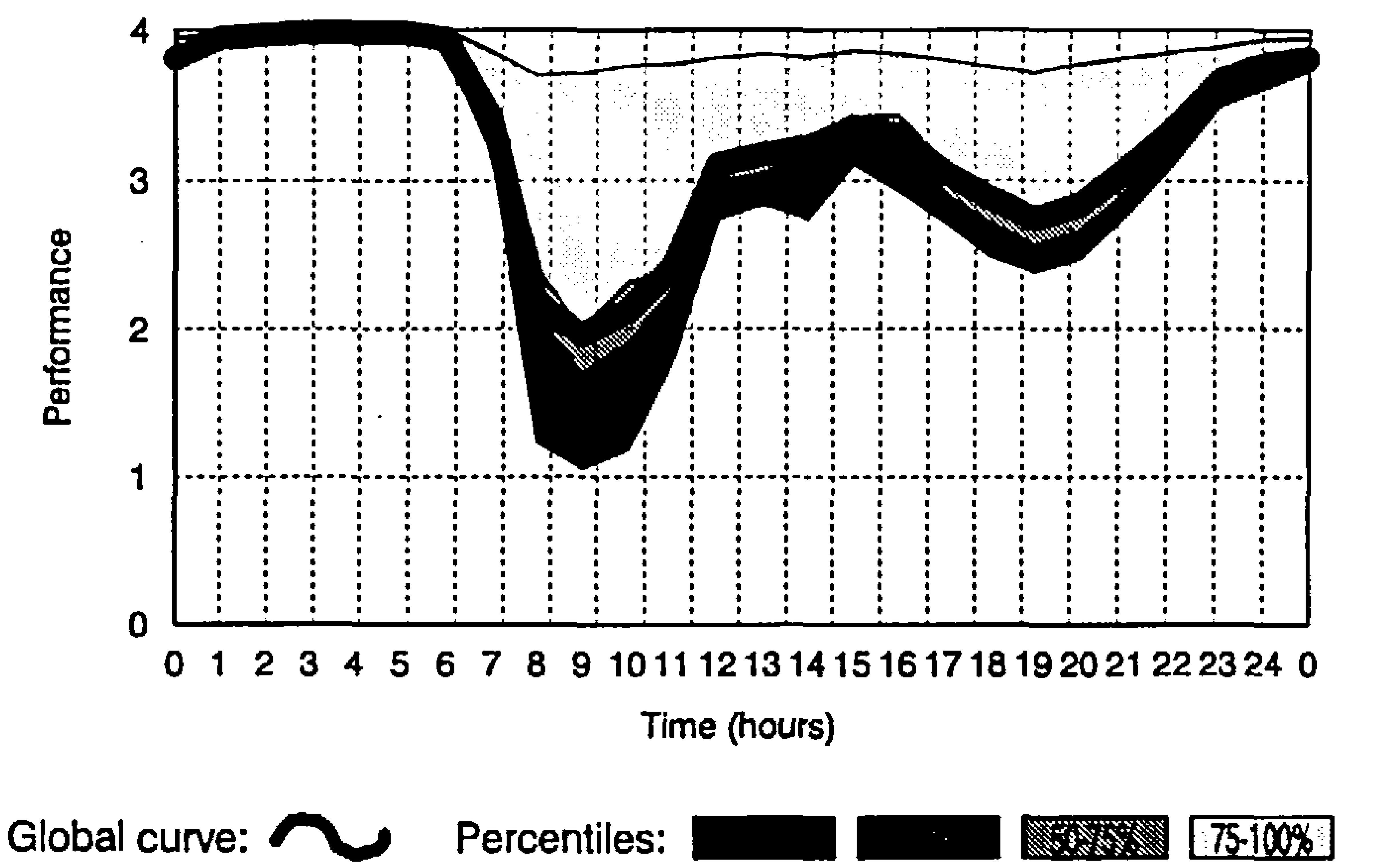

Fig.3.5 - An extended period graph with variation limits

Similarly, for an extended period simulation graph such as the one in figure 3.5, if *(t,y)* are the co-ordinates of a given point in the *P%* percentile curve, it means that at time *t*, *P%* of the total demand are being supplied with a performance index smaller or equal to *y*.

3.2.7. Graphical representation

Although the two types of curves just described contain some information about the distribution of values of the performance measures by means of the variation bands, they do not tell so much about the spatial variations that may occur within the portion of network that the curves aggregate. For that, it will be desirable to be able to depict the individual values of the indices at element level, if some sort of graphical representation of the network is available. Therefore, the method described here would best be front-ended by a network schematic facility showing a colour-coded representation of the performance indices at the network elements.

In actual fact, despite being a simple and obvious solution, network schematics and their colour-coding rather depend on the type of network analysis package used. For most cases, it is better done if programmed as an internal module of the package itself — in case it does have the necessary graphical capabilities — because it requires a spatial description of the network, either through the schematic or an actual digital map. Adding on those features at a post-processor stage is impractical, given the amount of spatial or geographical information that would have to be passed on from the simulator, beside the state variables.

Since no water distribution simulation package with easy access and manipulation of graphical information — namely network element co-ordinates — was available during the development of the present work, the computational effort required to achieve automatic network colour-coding was thought to be outside the scope of the work and therefore left for a future development of the suite of programs presented here.

3.3. COMPUTER PROGRAM PERF

The system for performance evaluation that is proposed and developed in the present work is materialised in a suite of programs that carry out all the necessary calculations for the production of the final graphical results.

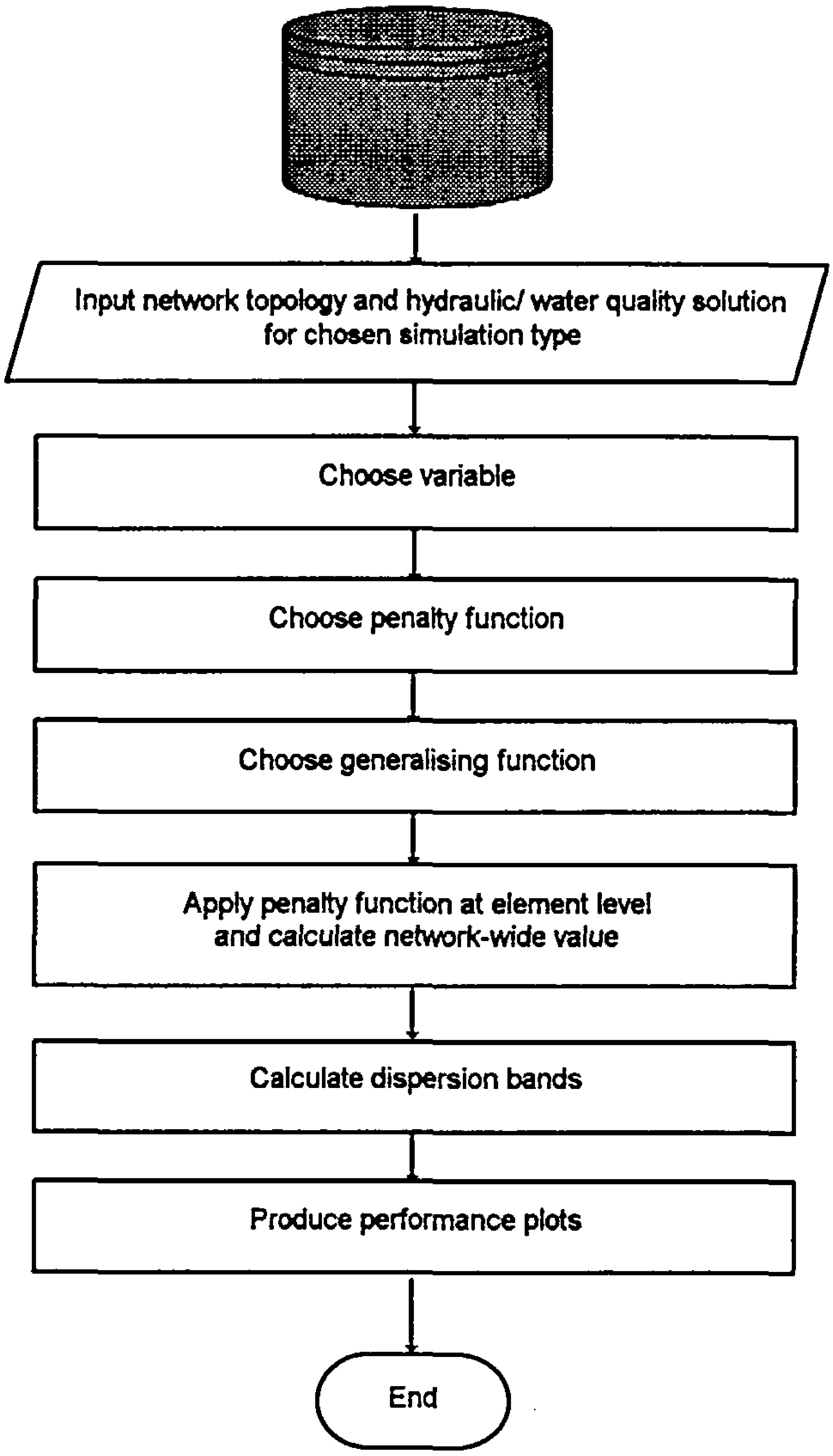

Fig.3.6 - Flowchart of PERF

The set of FORTRAN programs contemplate each of the domains where performance is evaluated in this work, as detailed in the next chapters, and is integrated in a systematic way through the main program PERF (Fig.3.6 shows the respective flowchart). This essentially implements the framework described in the previous sections by reading in the network's characteristics and modelled solution for the period concerned, establishing the three basic entities — variable, penalty function and generalising function — and calculating the performance values both at element level and globally. The performance plots are then produced in flat file format, ready to be taken by a graphical package.

3.4. SUMMARY AND CONCLUSIONS

A general methodology for the evaluation of technical performance of water supply and distribution systems is presented in this chapter. A systematisation of concepts helps establishing a standard approach to performance assessment that will be used as the common tool throughout the remaining chapters in this work. Standardisation is especially desirable in order to bring to the same quantified basis the various aspects that will be considered.

The method consists of choosing a state variable or network characteristic that quantifies the aspect relative to which performance is being assessed; a grading of the performance according to that variable; and a generalising function. The grading is translated through flexible penalty curves, which score the working range of values of the given state variable against a conventionalised system of performance gradings. The penalty curves are as much vehicles for common sense and level of service policy criteria as for the analyst's or engineer's sensitivity to a given aspect of a water network's behaviour.

The objectives set out in the opening section are achieved, of flexibility, standardisation and suitability for computational application through a numerical, quantitative approach. It is important, however, to bear in mind that the choice of state variable, indeed of performance measure, must above all contemplate network properties that can be standardised for the purpose of comparison, if not between different networks, at least between different operational demand scenarios.

The accuracy of the method is ultimately inherited from the source of the data it uses — mostly simulation results — and, conversely, its role can be seen as a synthetic analysis tool to avoid time-consuming examination of those data. The methodology has the potential to drive any of the currently used analysis and design processes, and was designed to be easily included in a simulation model.

The following chapters will apply this methodology to the three main areas of water system technical performance: hydraulics, water quality and reliability. The systematic approach presented here will be followed, with the identification of key aspects, their respective decision variables, the setting up of penalty curves and generalising functions, and the analysis of the resulting graphs for a variety of case studies.

CHAPTER 4

HYDRAULIC PERFORMANCE OF WATER DISTRIBUTION SYSTEMS

4.1. INTRODUCTION

The first area to be explored in an evaluation of a water distribution system's performance is inevitably its hydraulic behaviour. The processes of conceiving, designing, building and running a water supply system are first and foremost driven by the need to satisfy a given set of demand points with sufficient flow of water at usable pressure levels.

The main goals taken into account in the traditional approach to the design of water supply and distribution systems were the minimisation of initial investment and operational costs. The hydraulic performance was often relegated to a secondary role especially in terms of exploring how it can be affected by varying conditions throughout the life span of the system. The tendency has been reversed in recent years, with a new emphasis being placed on effective analysis of the way in which the system performs its water distribution task for a variety of circumstances. These are particularly important points in the case of urban distribution networks, often characterised by complicated layouts and myriad demand points but comparatively over-simplistic operation.

This type of problem is not exclusive to the design phases. It is common to find existing systems with under-design problems or functional and operational difficulties originating in hydraulic shortcomings. With the current rapid growth of many urban areas, there is a strong need for the type of tools that will allow the engineer and designer to evaluate the hydraulic performance of a system in order to facilitate diagnosis and decisions, without having to rely totally on the empirical insight of the experienced decision maker. The systematic use of network analysis models is certainly a correct path towards the solution of the problem. Water network simulators are an invaluable aid in the assessment of the system's response to alternative demand and operational scenarios. However, the type of results returned by such systems can be complex and far from intuitive, frequently making their interpretation difficult and less than objective when it is necessary to compare between different situations. That is where the present work proposes to act, by providing a standardised assessment of performance, which in this chapter will concern itself with the hydraulic behaviour of the network.

The present Chapter applies the performance assessment framework introduced previously to such hydraulic characteristics of water distribution systems as pressure head, velocity and headloss or energy consumption. As seen before, the performance evaluation framework establishes three types of entities for each network property or behavioural aspect it analyses: (i) A state variable which translates the said property at the *network element* level, from the point of view taken into consideration; (ii) a penalty function, mapping the values of the state variable against a scale of index values; and (iii) a generalising function, used for extending the element-level calculation across the network, producing zonal or network-wide values.

Since the whole system to be developed is based on state variables whose values are to be obtained through modelling of the network, it is important to highlight the main characteristics, uses and limitations of the existing network analysis methodologies. After introducing the subject of hydraulic modelling in supply and distribution networks, the present chapter reviews the main types of models, describes the general formulation governing the processes to be modelled and highlights the solving methods.

The selection of what hydraulic state variables to include in a performance evaluation system is then presented and measures concerning pressure, pressure fluctuation, flow velocity and energy consumption are proposed. The corresponding penalty curves and generalising functions are discussed, as well as the suitability of the various measures to the framework proposed and to the objectives of the work. Illustrative examples are given and the use and potential of the methodology explored.

4.2. HYDRAULIC MODELLING IN WATER DISTRIBUTION NETWORKS

4.2.1. Introduction

The hydraulic modelling of water distribution networks, for the purpose of network analysis and simulation, is carried out by solving a mathematical model that represents the physical components of a distribution system, the way they operate and the way in which they interact. With the advent of computers, these models became increasingly important as a flexible means of obtaining reliable estimates for the state variables of the network without actually resorting to exhaustive measurements of what are usually very complex systems.

Network analysis developed mainly throughout the nineteen-seventies and eighties, and is today considered a well established tool with a wide range of applications in planning, design, operation and management of water distribution utilities. The present section provides a brief overview of the main features and characteristics of this technique, whose principles, formulation and limitations constitute the stepping stone for much of the analytic work nowadays carried out in water supply and distribution, and form inevitably the basis for the present study.

4.2.2. Network analysis

A water distribution system can be represented as a network consisting of an interconnected set of nodes and links. The network nodes normally represent pipe junctions, pipe size changes or indeed any discontinuity in their characteristics, connections to special links such as pumps, valves and other pieces of equipment, measurement points, groups of consumers, particular spatial discretisation needs, etc.. In fact, a node is a conventional concept and can be inserted at any point of the network. Links, on the other hand, have a more physical meaning as they represent the actual components of the network, such as the aforementioned pipes, pumps and valves. Both demand and supply, the external forces that drive the network, are modelled as occurring at the nodes.

A model of a water distribution network does not necessarily have to include all of its pipes and elements. A complete system can frequently consist of so many pipes and connections that it makes it impractical to consider them all in one model, especially since it can be quite a task to actually find sufficient information about all those components in the first place. Simplification or *skeletonisation* of the network is commonly employed to reduce the size of the model, by discarding those pipes below a certain dimension, by lumping together groups of consumers or by replacing parts of the network with hydraulically equivalent pipes.

A network is described by its topology, which specifies the two end nodes of each link, as well as by the hydraulics-related characteristics of its components. These mean, in the case of the nodes, the elevation and the external supply and (or) demand flows. The links are normally described by the parameters of the hydraulic laws governing the flow through it, which in the case of pipes consist of length, cross-section and roughness coefficient.

Upon the physical description of the network itself, a hydraulic model consists of:

(i) The set of state variables necessary to describe the current state of operation. Those normally used for describing system states in modelling water distribution networks are nodal pressure and link flow.

(ii) A set of characteristic equations relating the state variables for each network element, such as the non-linear functions that relate flow to headloss in a pipe[1]. One of the most commonly used pipe flow equations is Hazen-Williams' approximation, where Q is the flow, C_{HW} the pipe roughness coefficient, D the pipe diameter, h the headloss per unit length and β a unit conversion factor[2]:

$$Q = \beta C_{HW} D^{2.63} h^{0.54} \tag{4.1}$$

(iii) A set of network governing equations, which aggregate the characteristic equations of all elements into the complete mathematical description of the network. The nodal mass-balance equations state that the sum of inflows at a node equals the sum of the outflows. For the general node i:

$$\sum_{k=0}^{U^i} q_{ki} - \sum_{j=0}^{D^i} q_{ij} = 0; \forall i \in N \tag{4.2}$$

Where q_{ij} is the flow[3] from i to j, and U^i and D^i the sets of upstream and downstream nodes of i.

A further set of equations translates the conservation of energy around network loops:

$$\sum_{l=1}^{NL^r} \Delta H_l = 0; \forall r \in NLP \tag{4.3}$$

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1 Equations for modelling the behaviour of the various network elements such as pipes, pumps and valves can be found, for example, in Walski (1984) or Twort et al. (1985).

2 For diameter in metres and flow in litres per second, the value of β is 278.534.

3 The notation for flows will follow the general rule that q_{ij} denotes flow in link ij, with 0 designating a supersource/supersink so that the fictitious links $0i$ and $i0$ may be respectively associated with the supply and consumption at node i. In this way, q_{0i}, q_{i0} are the supply and consumption flows at i, respectively.

ΔH_l is the headloss in link l, NL^r the number of links defining loop r and NLP the number of loops in the network.

The two sets of equations are in effect a pair of contragredient relationships, one written wholly in terms of link flow variables, the other wholly in terms of nodal heads, linked together by the element characteristic equations. The complete set of equations can be couched in terms of nodes, in terms of loops or both. The solution of the model yields the complete set of nodal pressure heads and link flow rates, which is known as the hydraulic solution.

There are several model solving techniques, of which the most prominent are: the Hardy-Cross method, using the loop equations (Hardy-Cross, 1936); the Newton-Raphson method using the loop equations (Martin and Peters, 1963, Ebb and Fowler, 1972); the Newton-Raphson method using the node equations (Shamir and Howard, 1968); the Linear Theory method, using both sets of equations (Wood and Charles, 1972); the Energy Minimisation method (Collins *et al.*, 1978); variations of the Newton-Raphson nodal formulation such as the Hybrid method (Carpentier *et al.*, 1985) or the Modified Gradient Method (Todini and Pilati, 1987). Discussion and comparison of the various methods can be found in Salgado *et al.* (1987), Germanopoulos (1988) or Nielsen (1989).

Finally, regardless of the performance of the solution methodology, it is important to recall that the mathematical model of a water distribution system is based on two main simplifying assumptions:

- demands are described as lumped at the nodes (various methods can be used for this aggregation, as discussed by Walski, 1984, and Alegre, 1986); and
- the model is usually but a skeletonised version of the real system.

There are models (Germanopoulos, 1988) that offer the possibility of modelling demands as a function of the available pressure head, but none has so far successfully addressed the stochastic nature of demands, both in time and space, which is one of the greatest limitations of the available methods (Alegre, 1992).

4.2.3. Steady-state models and dynamic models

The models described above can be used in single runs, in order to calculate the network hydraulic solution for one particular supply and demand scenario, or in a sequence of runs generated by a variation of the supply and/or demand conditions over a period of time. The first type of simulation is akin to a single snapshot of the network behaviour and uses what is termed a steady-state or static model. The second type would correspond to a sequence of snapshots as in a film, and is carried out by a so-called dynamic model performing extended-period simulations.

Steady-state models are mainly used to support the operation (namely for simulation of particular demand or configuration scenarios; definition of operational rules), maintenance (planning and simulation of maintenance works for minimal impact) and rehabilitation of the systems (diagnosis of shortcomings and simulation of remedial actions). Steady-state analysis is faster and easier to use than dynamic simulation, easier to calibrate and validate, and can realistically cope with almost any size or degree of complexity of the systems.

Dynamic simulation consists of a series of static solutions determined using the models described in 4.2.2., performed at pre-specified intervals, with the reservoir dynamics described by differential equations linking together two consecutive snapshots. Dynamic simulation is a powerful tool, in widespread use nowadays, whose correct application is complex and demanding. The increased calibration needs usually imply the existence of continuously monitoring equipment at critical points in the networks . It is better suited to lumped, macroscale representations than to highly detailed models of complex systems. Dynamic models are useful for supporting operational tasks, particularly the real-time control of large-scale systems and the definition of pumping or reservoir schedules, and planning and design tasks of systems with complex operation modes.

4.3. HYDRAULIC PERFORMANCE EVALUATION

4.3.1. Introduction

In Chapter 3, the basic requirements are laid out for the choice of a network property or state variable as a performance assessment measure within the methodology defined. The most important features are the relevance to the overall performance of the network and the potential for standardisation in the way proposed.

Three types of performance measure were reviewed in order to provide an evaluation of a system's performance from the hydraulic point of view. The three groups concern respectively pressure, velocity and energy considerations. The pressure and velocity measures result from traditional design and operational criteria and are relatively straightforward to relate to in terms of quantifiable performance. They were initially introduced by Alegre (1988) and Alegre and Coelho (1990; 1992), with further refinement by Alegre (1992) and Jowitt and Coelho (1994). The present section reduces them to the framework proposed in Chapter 3.

The possibility of a further measure based on energy consumption across the network is discussed. That will be followed by application examples that illustrate the use and potential of the methodology.

4.3.2. Pressure-related measures

The rationale for the pressure-related measures is based on the assertion that, for a network to perform well from a hydraulic point of view, the pressure at every supply point must fall between a maximum and a minimum requirement, and that the head surface across the network should not be subject to great fluctuations over time.

The minimum nodal pressure h_{min} is probably the single most important hydraulic requirement placed on the network, in order to meet the prescribed demands. Such a level is normally defined by the average height of the buildings that the water utility must supply without additional pressure boosting, and is typically in the order of 12 m to 20 m above that height. A further reason for wanting to keep pressures above safe minimum levels across the network is to stay clear of very low or negative pressures which may cause water quality problems (by drawing in outside water and material) or hydraulic instability.

The maximum pressure requirement h_{max} is set up according to the structural capabilities of pipes and other network elements. The main concern is related to leakage, one of the main problems currently faced by water managers, as progressively complex and older systems allow increasingly alarming quantities of water to be wasted underground. The leakage levels in any particular network are directly related to the pressure surface, and leakage control techniques (other than actual mains rehabilitation or replacement) are normally concerned with curtailing maximum pressures. On a less crucial note, excessive pressure also makes water difficult or less comfortable to use at the domestic tap.

To set up the pressure performance index following the methodology described previously, each node of the network is graded from 0 to 4 according to a penalty curve. Fig.4.1 shows an example of a conventional penalty curve for nodal pressure, translating what is probably a commonly established logic according to the following reasoning:

- The optimal nodal pressure is considered to be the one that equals the minimum pressure requirement h_{min}, since it is the best compromise between satisfying demand, minimising pumping costs and controlling leakage. Therefore, a node with such a pressure is graded at 4.

- A pressure value equalling the maximum allowable h_{max} is still meeting demand appropriately and is not expected to cause damage or a breakdown. Performance is therefore graded 3.

- Nodes where the pressure has exceeded in more than 50% the upper limit are thought to provide generally unacceptable service, and are penalised with a performance value of 1. The value of 2 will correspond to the intermediate stage.

- Finally, when the nodal pressure has fallen bellow 75% of the minimum requirement, there is no supply and the index takes the null value.

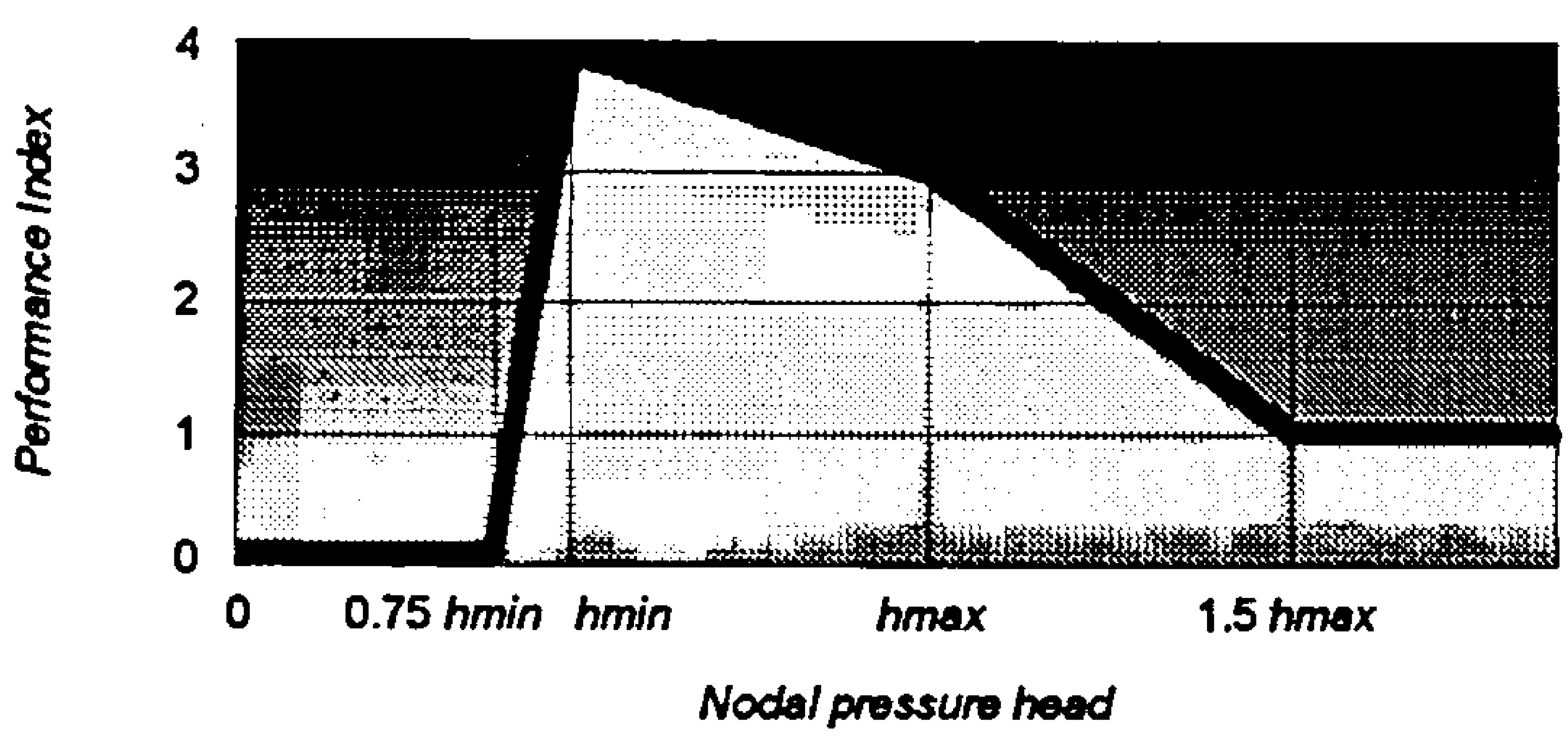

Fig.4.1 - Penalty curve for pressure

Typical values for the minimum and maximum nodal pressure requirements are respectively P_{min} = 20 m and P_{max} = 80 m. Most regulations[4] would probably require a less stringent lower

[4] In England and Wales, for example, this limit is established by the Office of Water Services at 10 metres, for a flow of 9l/min at the consumer's stoptap.

minimum requirement, but in practise water utilities prefer to self-impose a higher threshold to stay clear of any infringements to the legal limit and to guarantee a better service.

~ ~ ~

On the other hand, significant fluctuations of the head surface throughout the routine operation of a system are unwanted both because they mean inconvenience for the user and because they are generally associated to a greater sensitivity of the system to small changes in the operational scenario.

A head fluctuation measure can be set up to evaluate the hydraulic performance of a system from the point of view of the fluctuations in the head surface during daily operation or for any other given variation range of its operating conditions. In this case, a range of conditions does have to be specified as each value of the performance measure is calculated by comparison with the maximum pressure found at the particular node.

As with the pressure measure seen before, the calculation is done on a nodal basis and the following penalty curve, applicable to the difference between the maximum pressure and the measured pressure at the node, is proposed:

- The optimal state would be to keep a node always at constant pressure, regardless of demand and operating conditions. This situation is graded at 4.

- A reference maximum nodal head fluctuation Δh_{max} must be defined, to which the performance value of 1, i.e., the unacceptability threshold, is assigned. Water distribution legislation and company practice normally consider a figure of 30 to 40 m as such a maximum allowable variation.

- The two points thus defined generate a linear penalty function as shown in Fig.4.2.

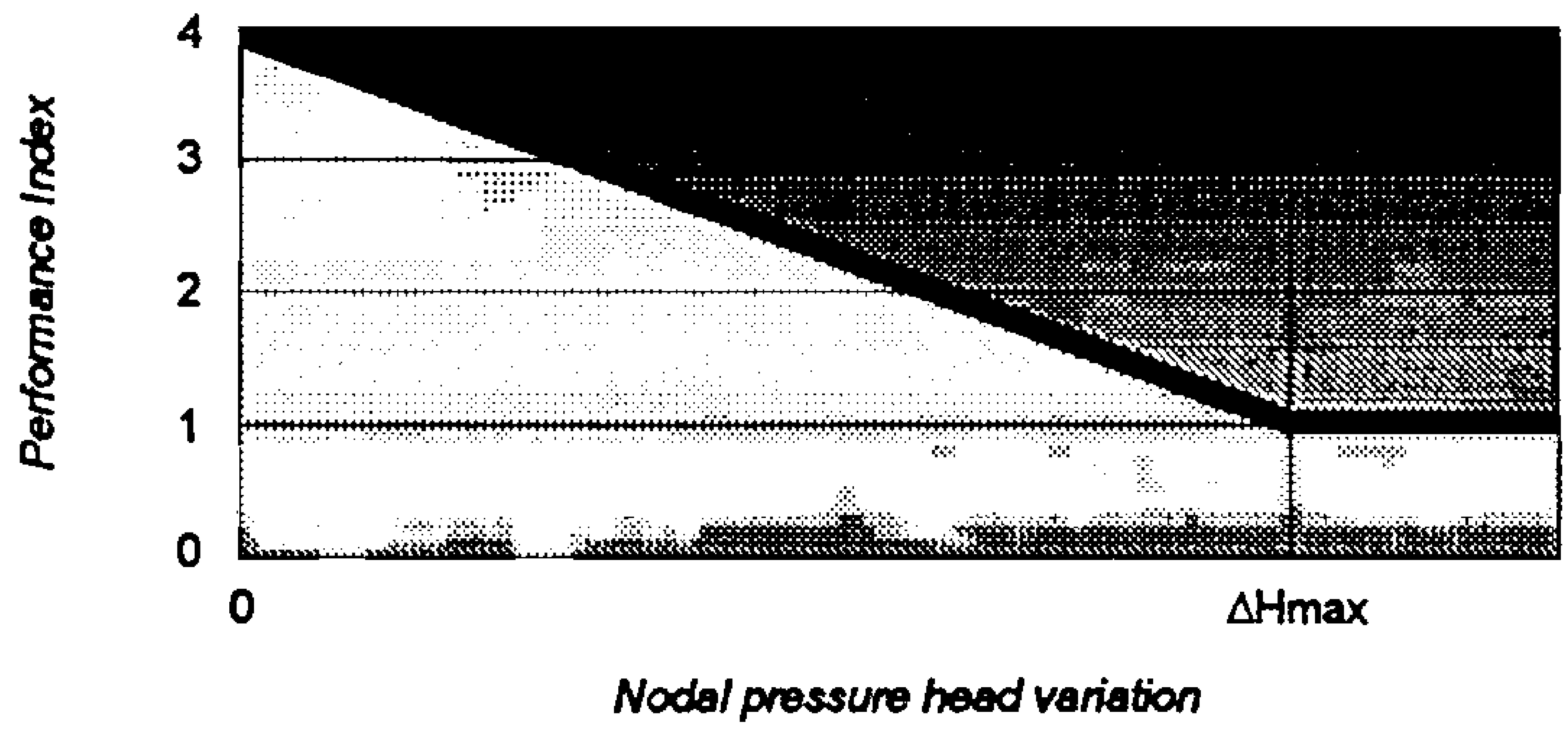

Fig.4.2 - Penalty curve for pressure fluctuation

In terms of generalising function for both pressure-related performance measures, it makes sense to calculate an average-based value to represent the network. However, since both measures are mostly consumption-oriented, it is important to weigh each nodal value in terms of how much demand it affects. For that reason, nodal demands are used as weights in the following expression for the generalising function:

$$\mathrm{P} = \mathrm{W}[pm_i] = \sum_{i=1}^{N} w_i pm_i \tag{4.4}$$

Where P is the global value of the performance index, pm_i the value of the index at node i and w_i the nodal weights, given by the fraction of total consumption:

$$w_i = \frac{Q_i}{\sum_{i=1}^{N} Q_i} \tag{4.5}$$

4.3.3. Velocity-related measures

The approximately quadratic relationship between velocity and headloss means that variations in the head surface are associated with flow velocities. On the other hand, most network designers and managers prefer to keep a check on flow velocities. Very high values may have negative structural consequences on the pipes, or represent extreme and undesirable hydraulic regimes. On the other hand, very low velocities may induce excessive deposit or even stagnation of flow, with the implications that may have for the quality of the water. In some countries, a reference velocity V_{ref} is used as a design criterion and calculated according to the size of the pipe and its expected maximum flow capacity, such as in the following expression (Baptista, 1983), given for a link of diameter D :

$$V_{ref}\,(\text{m/s}) = 0.1274\, D^{0.4}\,(\text{mm}) \tag{4.6}$$

The second type of hydraulic performance index is therefore a velocity measure for link flow (the link being the network element as defined in Chapter 3). It is based on a simple but plausible classification, developed from the perception of water network managers according to the following penalty curve:

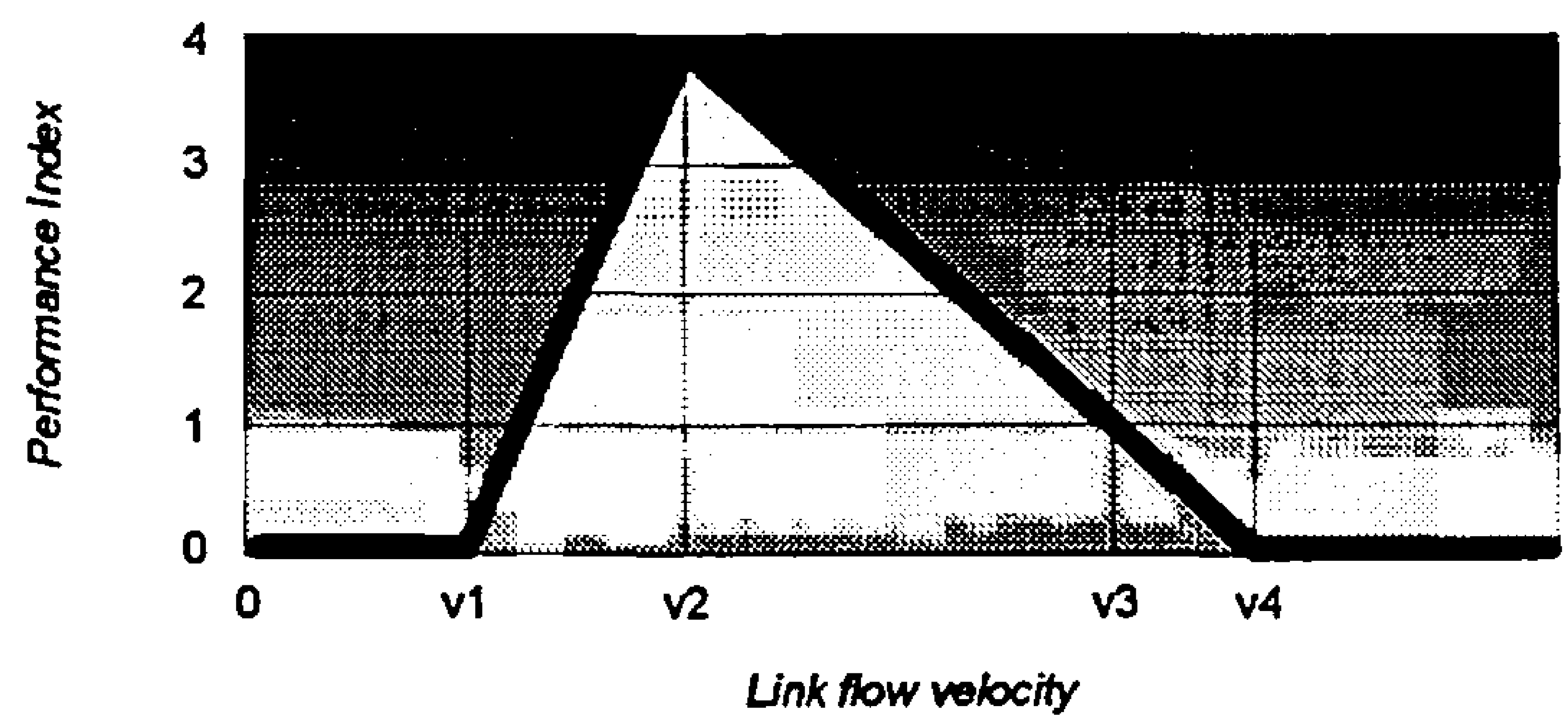

Fig.4.3 - Penalty curve for velocity

The suggested reference velocity is V_{ref}= 0.5 m/s. In the diagram, the following values are used:

$$V_1 = 0.5\, V_{ref} \qquad V_2 = V_{ref} \qquad V_3 = 2.0\, V_{ref} \qquad V_4 = 3.0\, V_{ref}$$

The generalising function to be applied in this case is less intuitive than in the pressure-based nodal measures. Again a weighted average seems correct, but what weights to use may be arguable. The weighing factor must make the result independent from the choice and placement of nodes, since these are merely a convention as seen before. That would be achieved using the pipe length. However, pipe size (cross-section) must also be taken into consideration, since a larger mains is implicitly more important than a small diameter pipe, for the same unit length. To incorporate both the length and the cross-section, the pipe volume was chosen:

$$\mathrm{P} = \mathrm{W}[pm_r] = \sum_{r=1}^{NL} w_r pm_r \tag{4.7}$$

Where P is the global value of the performance index, pm_r the value of the index in link r, NL the number of links in the network and w_r the link weight, given by the pipe volume:

$$w_r = L_r \pi \frac{D_r^2}{4} \tag{4.8}$$

The velocity index must be analysed in conjunction with the pressure measures, since if taken alone its significance is relatively poor. The index was calibrated through experimentation in over 15 networks (Alegre and Coelho, 1992), in order to match it to the behaviour of the head fluctuation index. If well calibrated, comparing the two can provide a rough idea of how favourable or unfavourable the topology of the system may be, on average terms. Velocity indices below their head fluctuation counterparts may indicate a bad configuration, and vice-versa. However, since the networks used for calibration are not exactly a representative random sample of the existing population, such calibration must be taken with appropriate caution. Only the continued application to real case studies will allow for more reliable values.

4.3.4. Energy-related measures

The subject of energy dissipation around the network is not as clear-cut a domain for performance evaluation as the previous two, but it may be worthwhile nevertheless to explore the possibilities that may be available using the present methodolpgy. If nothing else, it may help to understand a little better the relationships that may be established between the pressure and velocity measures.

Energy calculations in water supply networks are better couched in terms of power: the power *Pw* dissipated in order to carry the flow *Q* consumed at a given node from its supply source elsewhere in the network is given by the difference in pressure head, or headloss, between the two nodes, multiplied by the flow:

$$Pw_{(w)} = \Delta h_{(Pa)} \cdot Q_{(m^3/s)} \qquad (4.9)$$

The difficulty is that, for the general network, a given flow or consumption may originate from more than one source. A fast and efficient solution can be found, however, using the formulation developed by Xu (1990) (see also Jowitt and Xu, 1993). The distributions of flow components by source or by destination, also called *microflow* distributions, can be calculated using only mass-balance considerations, assuming fully-mixed flow at the nodes, without the need to undertake full network simulations.

For the generic node *i*, with a set U^i of upstream nodes and a set D^i of downstream nodes for the particular network flow distribution, incident flows are denoted by q_{ki}, including the source inflow q_{0i}, and emergent flows by q_{ij}, including the consumption q_{i0}. Furthermore, ${}^{gh}q_{ij}$ indicates the proportion of flow in link[5] *ij* originating in link *gh*, and q_{ij}^{kl} refers to the proportion of flow in link *ij* which is destined to flow in link *kl*.

The microflows formulation can be couched in terms of composition of a given consumption flow or link flow by source. The proportion of consumption at node *i* originating from the

[5] It must be borne in mind that "link" is taken here in the broader sense, including therefore the aforementioned fictitious links 0*i* and *i*0 that represent external supply and demand at node *i*.

source supply at node m is denoted by ${}^{0m}q_{i0}$, and is given, for $i \neq m$ and for $i=m$, respectively by:

$$ {}^{0m}q_{i0} = \frac{\sum_{k \in U^i} {}^{0m}q_{ki}}{\sum_{k \in U^i} q_{ki} + q_{0i}} q_{i0} \tag{4.10} $$

$$ {}^{0m}q_{m0} = \frac{q_{0m}}{\sum_{k \in U^m} q_{km} + q_{0m}} q_{m0} \tag{4.11} $$

The proportion of flow q_{ij} that originates from $0m$, the source at node m, is denoted by ${}^{0m}q_{ij}$, and is calculated thus, for $i \neq m$ and for $i=m$ respectively:

$$ {}^{0m}q_{ij} = \frac{\sum_{k \in U^i} {}^{0m}q_{ki}}{\sum_{k \in U^i} q_{ki} + q_{0i}} q_{ij}; \quad \forall j \in D^i \tag{4.12} $$

$$ {}^{0m}q_{mj} = \frac{q_{0m}}{\sum_{k \in U^m} q_{km} + q_{0m}} q_{mj}; \quad \forall j \in D^m \tag{4.13} $$

The sequential application of these formulae, working from sink to source nodes throughout the network, produces the complete set of proportions of consumption flow and link flow originating in each source.

The power dissipated in a network in order to supply the demand at node i, Pw_i^{diss}, is given by an equation analogous to 4.9:

$$ Pw_i^{diss} = \sum_{s=1}^{NS} (h_s - h_i).{}^{s}q_{i0} = \sum_{s=1}^{NS} \Delta h_{si}.{}^{s}q_{i0} \tag{4.14} $$

${}^{s}q_{i0}$ being that part of the demand at node i originating in source s, NS the number of sources, and $h_s, h_i, \Delta h_{si}$ respectively the head at s, the head at i and the headloss between s and i.

The total power dissipated in a network in order to supply all the demanded flows, Pw^{diss}, is given by:

$$Pw^{diss} = \sum_{i=1}^{N}\sum_{s=1}^{NS} \Delta h_{si} \cdot {}^{s}q_{i0} = \sum_{i=1}^{N} Pw_i^{diss} \tag{4.15}$$

N being the number of nodes.

On the other hand, it is quite straightforward to express the minimum power[6] needed to satisfy the minimum nodal pressure requirement at a node i in the following manner:

$$Pw_i^{\min} = h_{\min} \cdot q_{i0} \tag{4.16}$$

The minimum power to satisfy the minimum pressure requirement in all nodes across the network is:

$$Pw^{\min} = \sum_{i=1}^{N} h_{\min} \cdot q_{i0} = \sum_{i=1}^{N} Pw_i^{\min} \tag{4.17}$$

Equally, the power needed to supply node i at service pressure h_i:

$$Pw_i = h_i \cdot q_{i0} \tag{4.18}$$

And the total in all nodes across the network is:

$$Pw = \sum_{i=1}^{N} h_i \cdot q_{i0} = \sum_{i=1}^{N} Pw_i \tag{4.19}$$

The surplus power available when supplying node i is, on the other hand:

$$Pw_i^{sur} = (h_i - h_{\min}) . q_{i0} = Pw_i - Pw_i^{\min} \tag{4.20}$$

While the total surplus power made available to the network is:

6 In case of gravity-fed systems, this is the equivalent power or *potential* power.

$$Pw^{sur} = \sum_{i=1}^{N} (h_i - h_{\min}).q_{i0} = \sum_{i=1}^{N} Pw_i^{sur} = Pw - Pw^{\min} \tag{4.21}$$

All these power quantities may of course be divided by the corresponding flows to yield power per unit flow.

A few observations can be made with respect to the above quantities. Thus, the minimum power $Pw^{\min}$ is a basic requirement which the engineer has to comply with, and can do very little about. On the other hand, the surface defined by $Pw_i^{\min}$ will always constrain the values of surplus power Pw_i^{sur} supplied to the system, since all the flow originating from a particular source at any given moment is supplied at the same pressure head. In order to satisfy Pw_i^{sur} at the least favourable node, surplus power will be made available at all the other nodes. Here, the engineer can already intervene, by trying to design in order to minimise those discrepancies. It is however important to have some knowledge of Pw^{diss}, Pw_i^{diss}, in order to guarantee that the minimisation of those discrepancies is not achieved by simply dissipating the surplus power.

Whatever the objectives, it seems worthwhile to try and gain some sensitivity to the variation of the above quantities, and the performance assessment framework developed could be used for that purpose. The main obstacle, however, to establishing a measure of performance in this area is the lack of an accepted standard, such as the minimum requirement for pressures, that would allow for straightforward comparison between networks or between different configurations of the same network. It is possible, in the meantime, to establish system curves such as defined before, utilising absolute values instead, to provide an idea of the potential of the system or the way its power consumption properties can be expected to vary within a range of demand loads. This will be exemplified subsequently (section 4.4.5).

4.4. APPLICATION EXAMPLES

4.4.1. Introduction

The methodology previously described is now illustrated by means of some application examples. The first 3 examples are based on existing water distribution systems in England: Town A, a small rural town in hilly countryside; Town B, a slightly bigger mixed rural/suburban area; and Town C, a large seaside suburban town in very flat terrain. They cover an interesting range of network configurations, operational scenarios and demand behaviours, and usefully illustrate the general application of the methodology. The models used shared the actual, fully calibrated and updated Watnet[7] source files that the water company concerned utilises for the operation and management of the systems. These examples are used for generally illustrating the various aspects of the methodology in terms of knowledge gain to the hydraulic behaviour of the system.

A fourth example illustrates some considerations regarding the aforementioned analysis of power use and dissipation.

4.4.2. Town A distribution system

Town A is a residential area in a rural setting, with a distribution network serving about 9 000 people in a relatively hilly topography. The simulation model (schematic shown in Fig.4.4) comprises 50 nodes, 60 links, 2 variable level reservoirs, 2 time-switched pumps and 1 non-return valve. The network data are given in Appendix A.

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[7] The educational version of WRc's Watnet 4.0 package was used in this study for general network analysis and simulation.

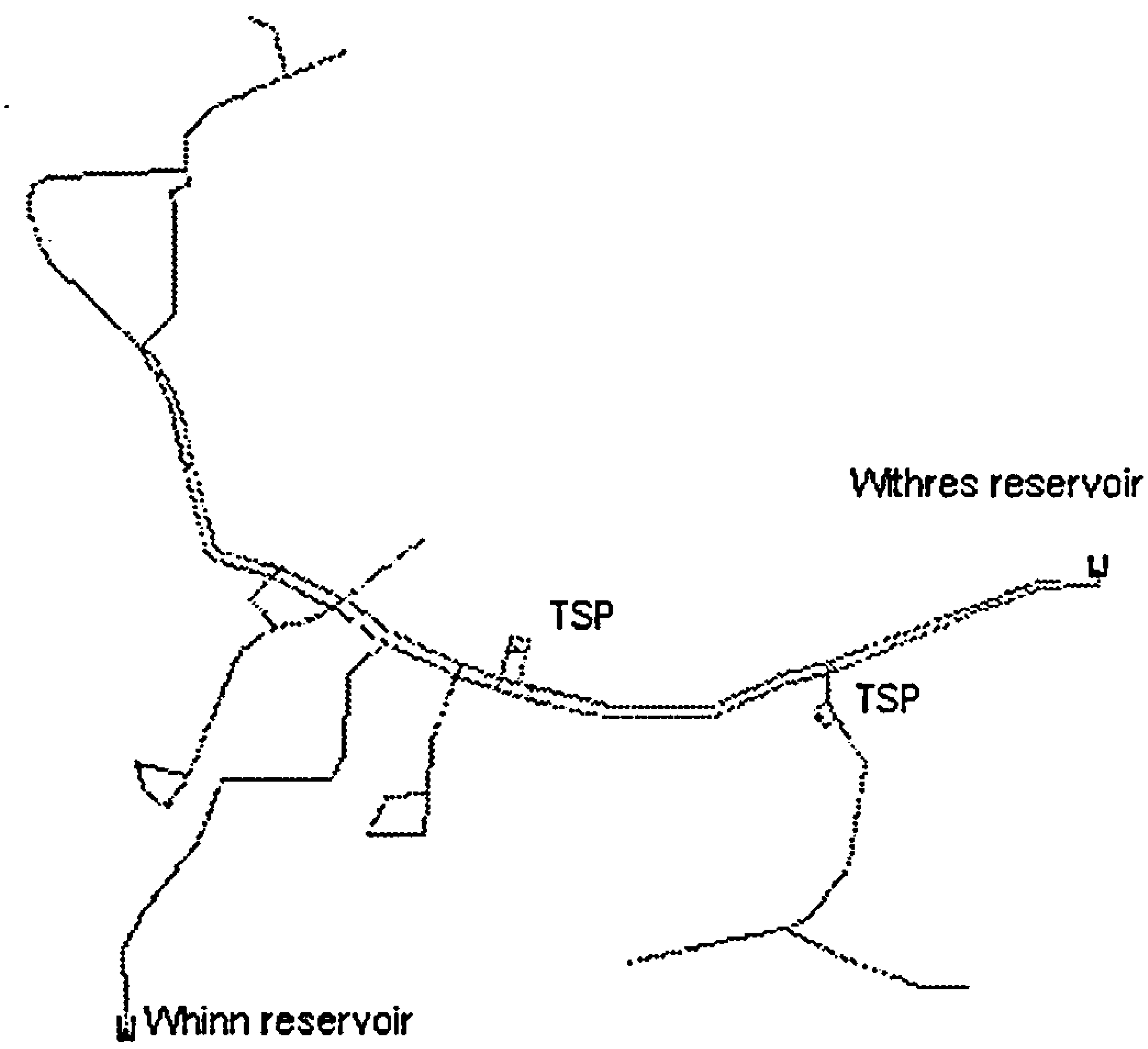

Fig.4.4 - Town A network schematic

System analysis diagrams

The diagrams displaying the variation of the three performance indices for the system analysis are shown in Fig.4.5. Starting with the pressure index, it can be seen that the index curve follows a relatively typical pattern, with an ascending stretch in the upper values of the index levelling out smoothly before a descending limb that eventually drops to less acceptable

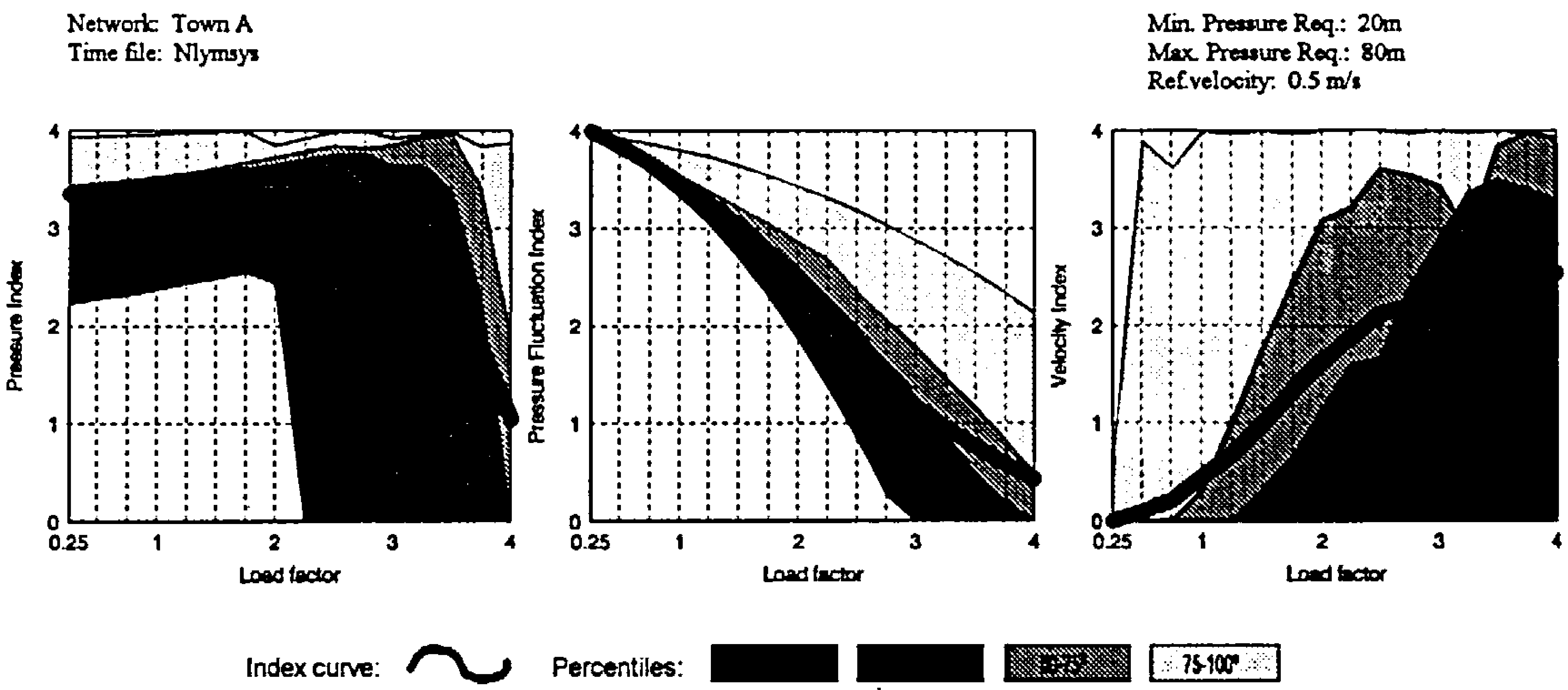

Fig.4.5 - Town A system performance diagrams

values.

The ascending limb corresponds to a range of loads for which the system is operating with pressures above the recommended minimum (briefly looking at the pressure index definition diagram on Fig.4.1, the index values are to the right of the peak, working their way uphill as demands increase and available heads decrease). The curve remains above 3 in this first stretch, which means that those pressures are still adequate and not excessive. That, however, is considering only the mean curve. Taking a look at the dispersion bands, it can be seen that part of the lower percentile (around 10-20% of the demand) is indeed below 3, corresponding to nodes with excessive pressure head.

As the demands keep increasing (and pressures decreasing) the pressure index values go past the optimum and start falling (to the left in the pressure index definition diagram). This causes the system index curve to level out and eventually start falling, when the available heads drop below the minimum required. In the case of Town A, the index curve for the system as a whole remains at very acceptable levels for loads up to 2.75 times the average. However, there are demand nodes in clear difficulty for loads above 2, as can be seen from the two lower dispersion bands. Above 3.6 the system no longer provides an acceptable level of service.

The relatively narrow bands indicate a fairly homogeneous system up to a load of around 2. However, the hump on the index curve between 3 and 3.75 would appear to be caused by two areas with different behaviours at those load levels.

It is interesting to notice that the index curve drops below the 50% percentile for loads of 2 and above, corresponding to a skewed distribution of index values across the demand nodes domain. This will possibly mean that the problems may be localised or due to a small number of demand nodes.

Turning now to the pressure fluctuation index, it shows a smooth typical behaviour, with acceptable values for loads of up to around twice the average. At this level the dispersion is still well contained and the system seems fairly homogeneous. The slope of the curve shows that the system will be moderately sensitive to changes in the load.

As for the velocity index, its steady growth throughout the whole domain means that, with respect to this criterion, the system is over designed, i.e., it displays low velocities for the entire range. Only above loads of 2.25 does the performance become acceptable on average, but the width of the dispersion bands reveals great heterogeneity.

24 hour simulation diagrams

The diagrams showing the variation of the three performance indices over a 24 hour extended period simulation for the Town A network are shown in Fig.4.6 to Fig.4.8.

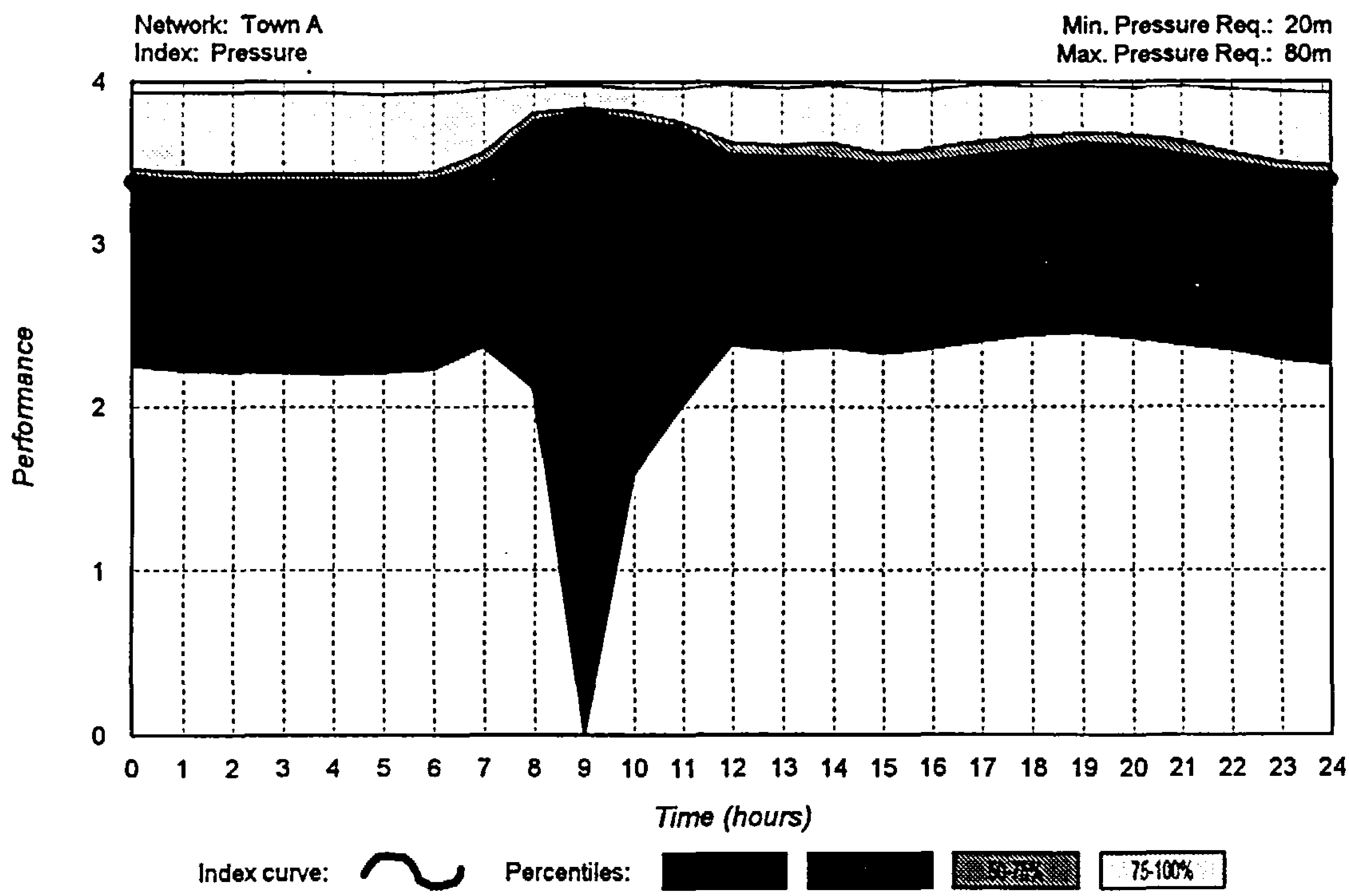

Fig.4.6 - Town A extended-period performance simulation for pressure

The pressure index shows an acceptable performance throughout the 24 hour period. Only the lower percentile band falls below 3, but the fact that its width is much greater than any of the other bands, especially the intermediates, probably means that the problem nodes are few and that localised changes in the system may significantly improve its performance. The sudden drop at around 09:00 is due to low pressures occurring at a small number of nodes.

The pressure fluctuation index displays a less acceptable behaviour during the morning working period, with the average curve bordering on a pressure fluctuation index of 2 and 75% of the demand below that value. This is probably explained by the network's topography, which is far from flat, and by the fact that it is partially supplied by another network (modelled through the WHINN variable head reservoir) with a significant head fluctuation of over 20 m throughout the day.

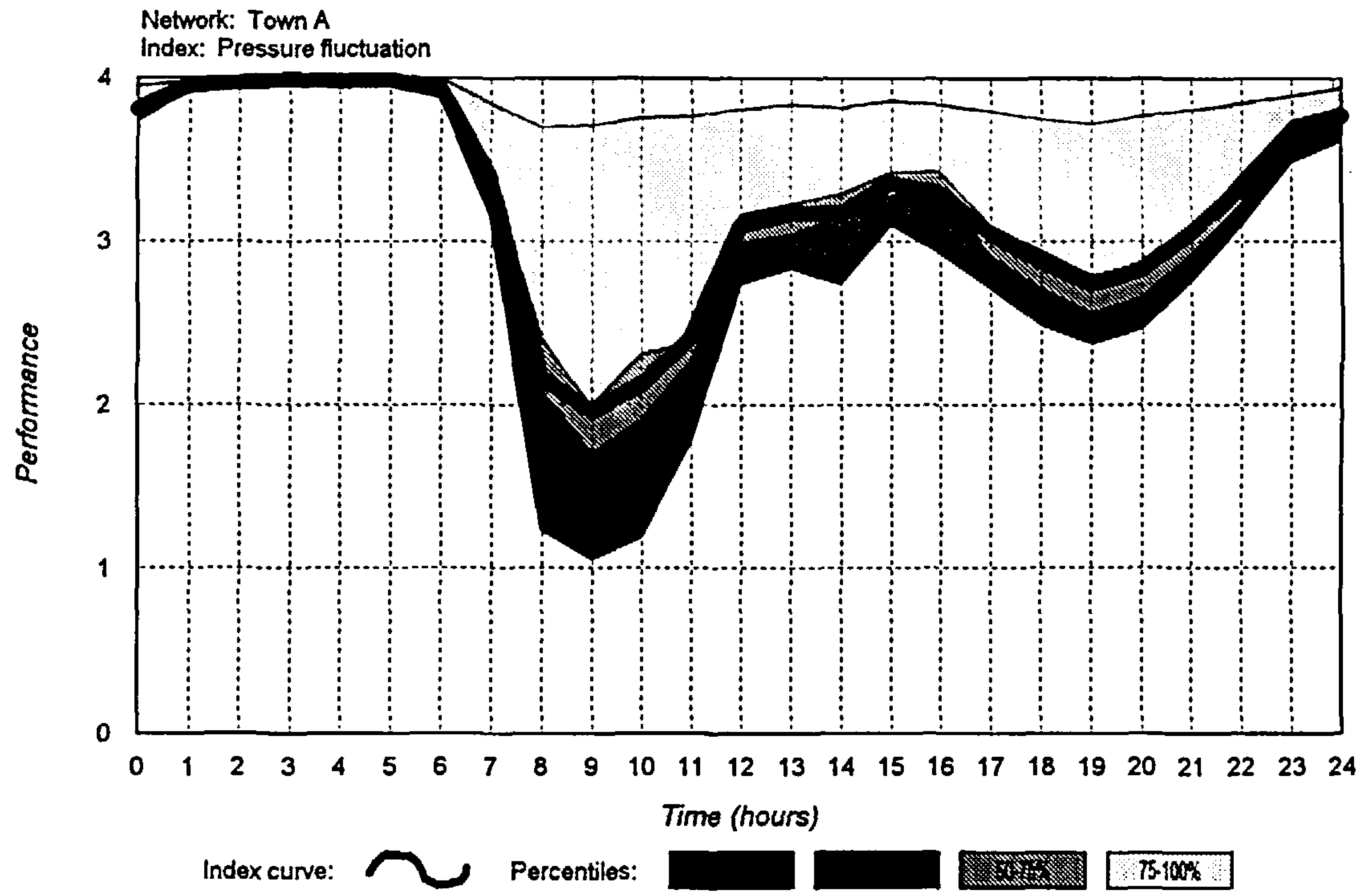

Fig.4.7 - Town A extended-period performance simulation for pressure fluctuation

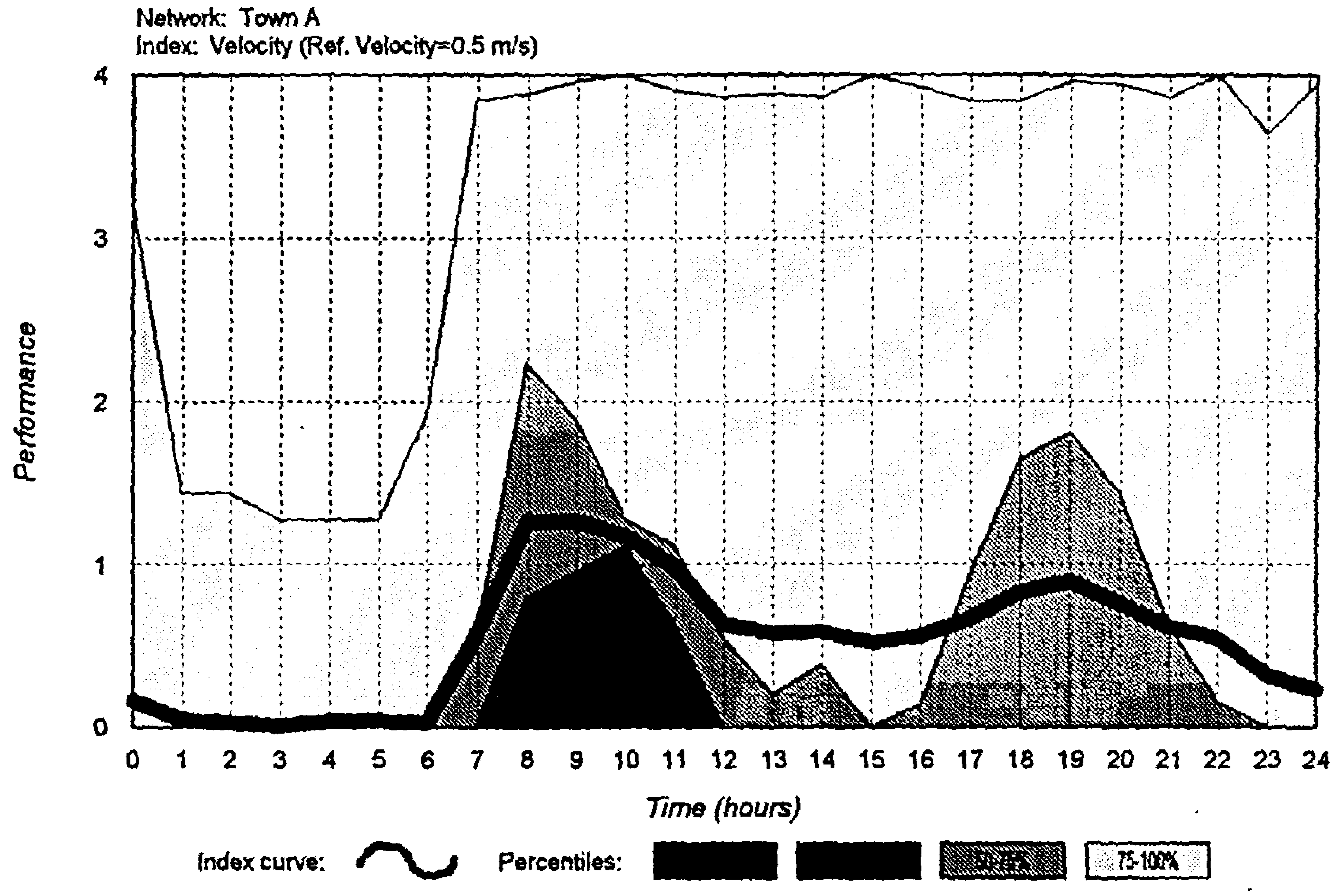

Fig.4.8 - Town A extended-period performance simulation for velocity

The velocity index merely translates what would already be expected from the behaviour of the system diagram across the equivalent demand load range. In particular, it shows that night-time velocities are seriously low across the system, which may induce stagnation and sedimentation problems.

4.4.3. Town B distribution system

Town B is a mixed rural/suburban area, with a distribution network (Fig.4.9) feeding approximately 15000 people and a gently varying topography with a difference of about 30 metres between highest and lowest point. The system comprises 164 nodes, 198 pipes, 2 variable head reservoirs, 1 fixed head source, 3 time-switched pumps and 4 non-return valves. The network data are given in Appendix A.

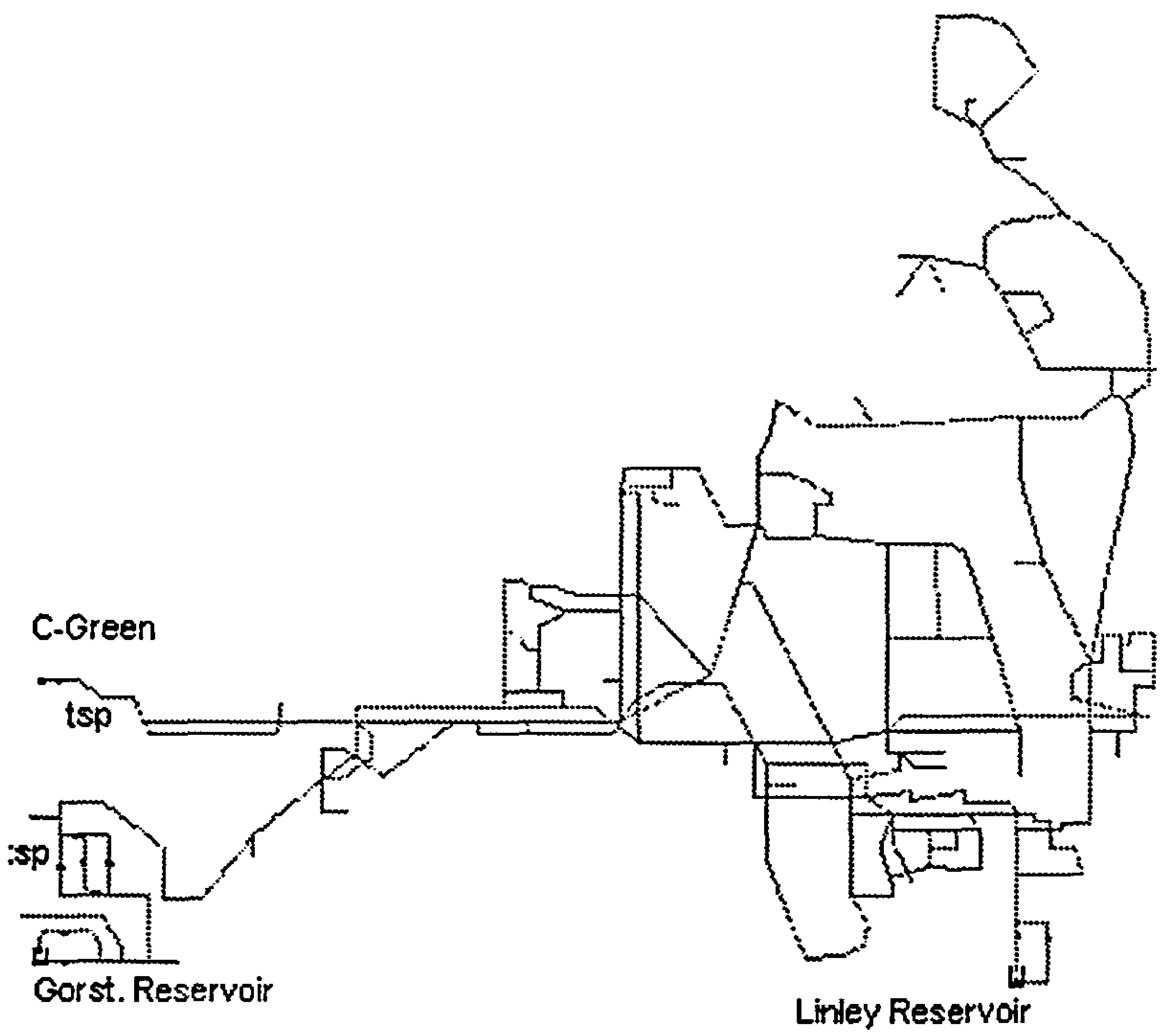

Fig.4.9 - Town B network schematic

System analysis diagrams

The system analysis diagrams for Town B, in Fig.4.10, are relatively similar to those for Town A. The steeper pressure fluctuation index curve indicates that the system is very

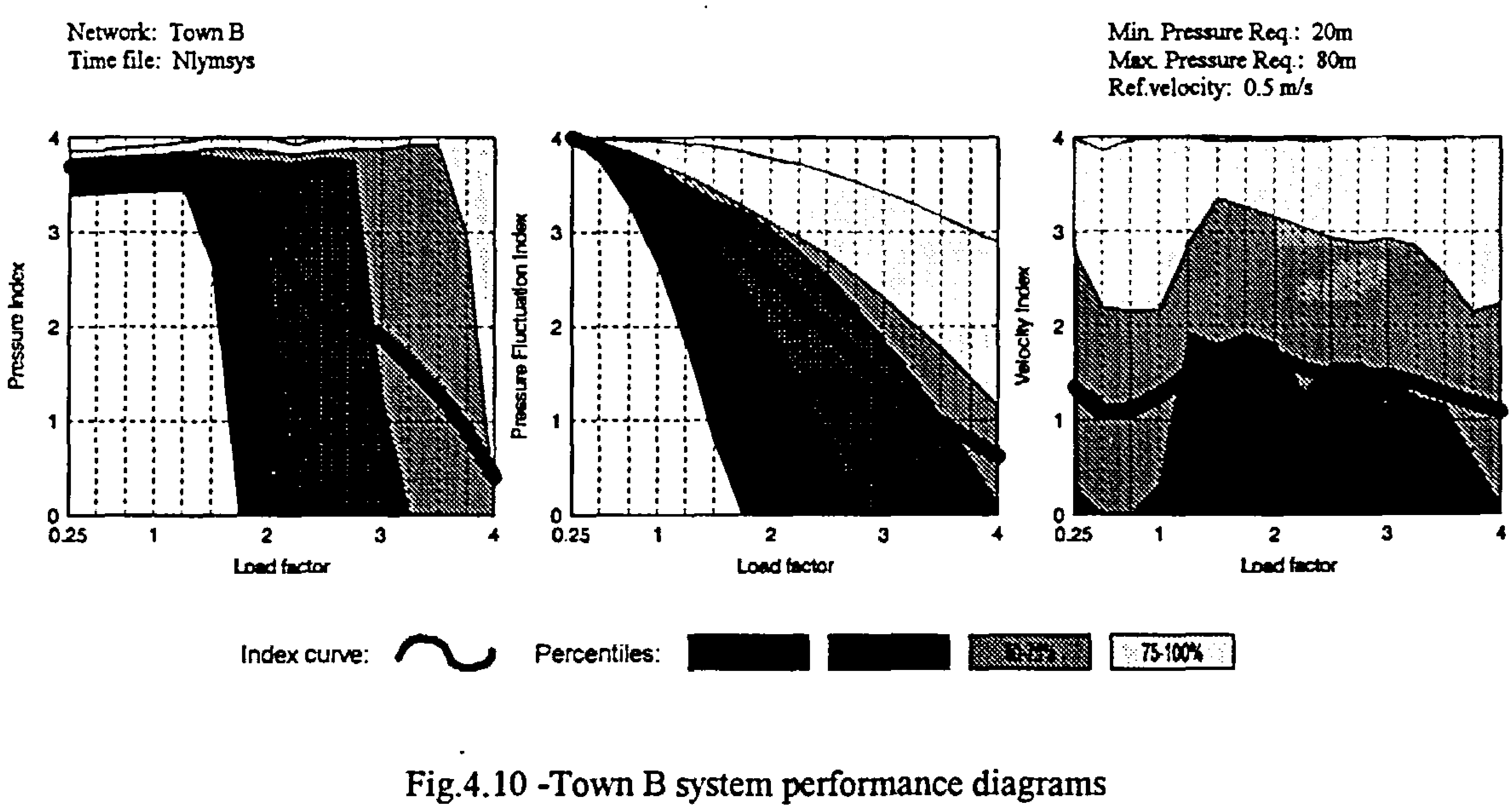

Fig.4.10 -Town B system performance diagrams

sensitive to load variations – this can make it difficult to control and operate. The velocity index curve has a flat, less sensitive response, but the values are below the acceptable level as defined by the reference velocity adopted. The shape of the diagram, increasing at first, then decreasing without even reaching 2, together with the width of the dispersion bands, will probably be due to the existence of both too low and too high velocities in what is likely to be a heterogeneous system from this point of view.

24 hour simulation diagrams

The diagrams showing the variation of the three performance indices over a 24 hour extended period simulation for the Town B network are shown in Fig.4.11 to 4.13.

For the load variation throughout the day, the Town B network performs well as far as the pressure index and pressure fluctuation index diagrams are concerned, with a less satisfactory performance on the velocities side.

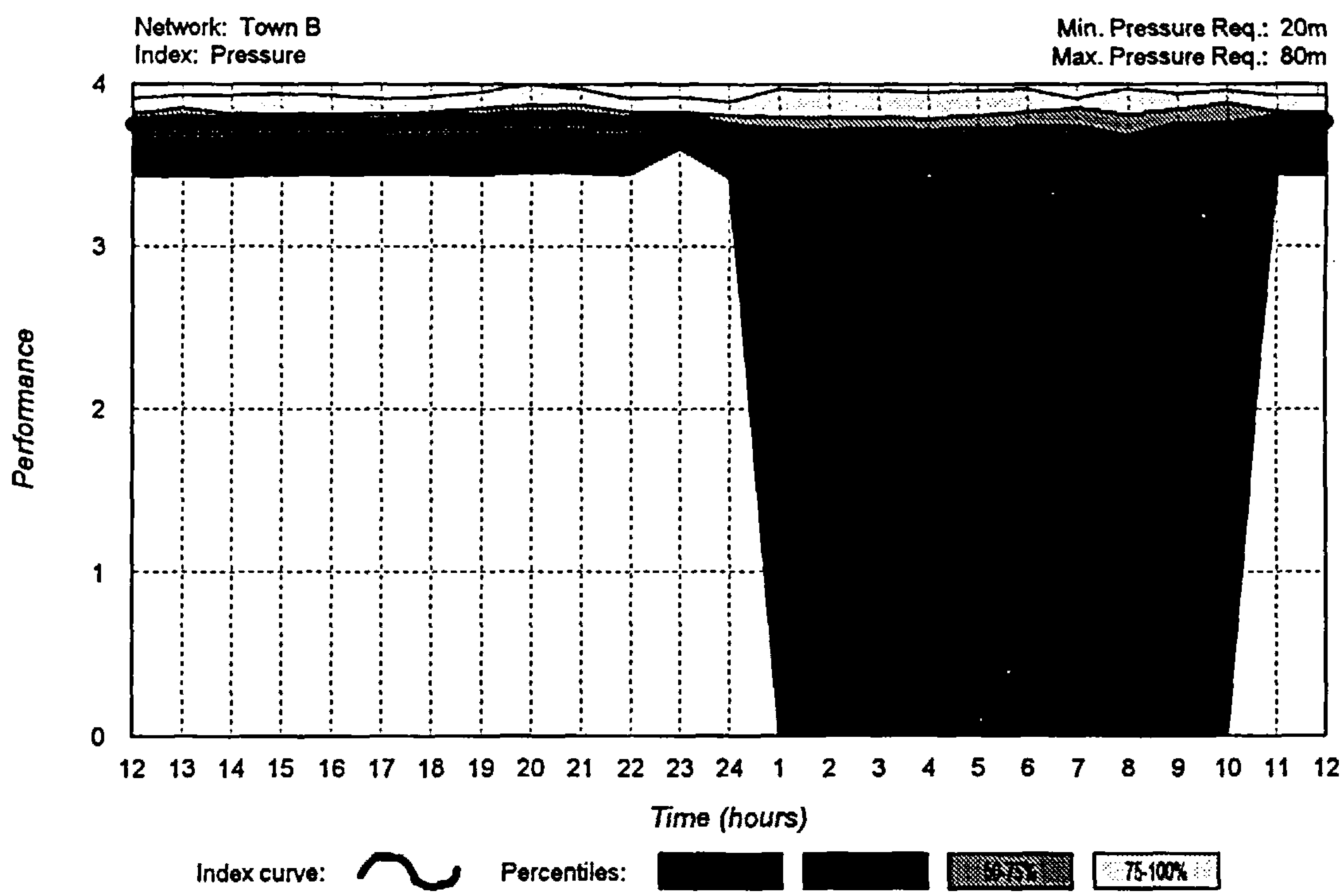

Fig.4.11 - Town B extended-period performance simulation for pressure

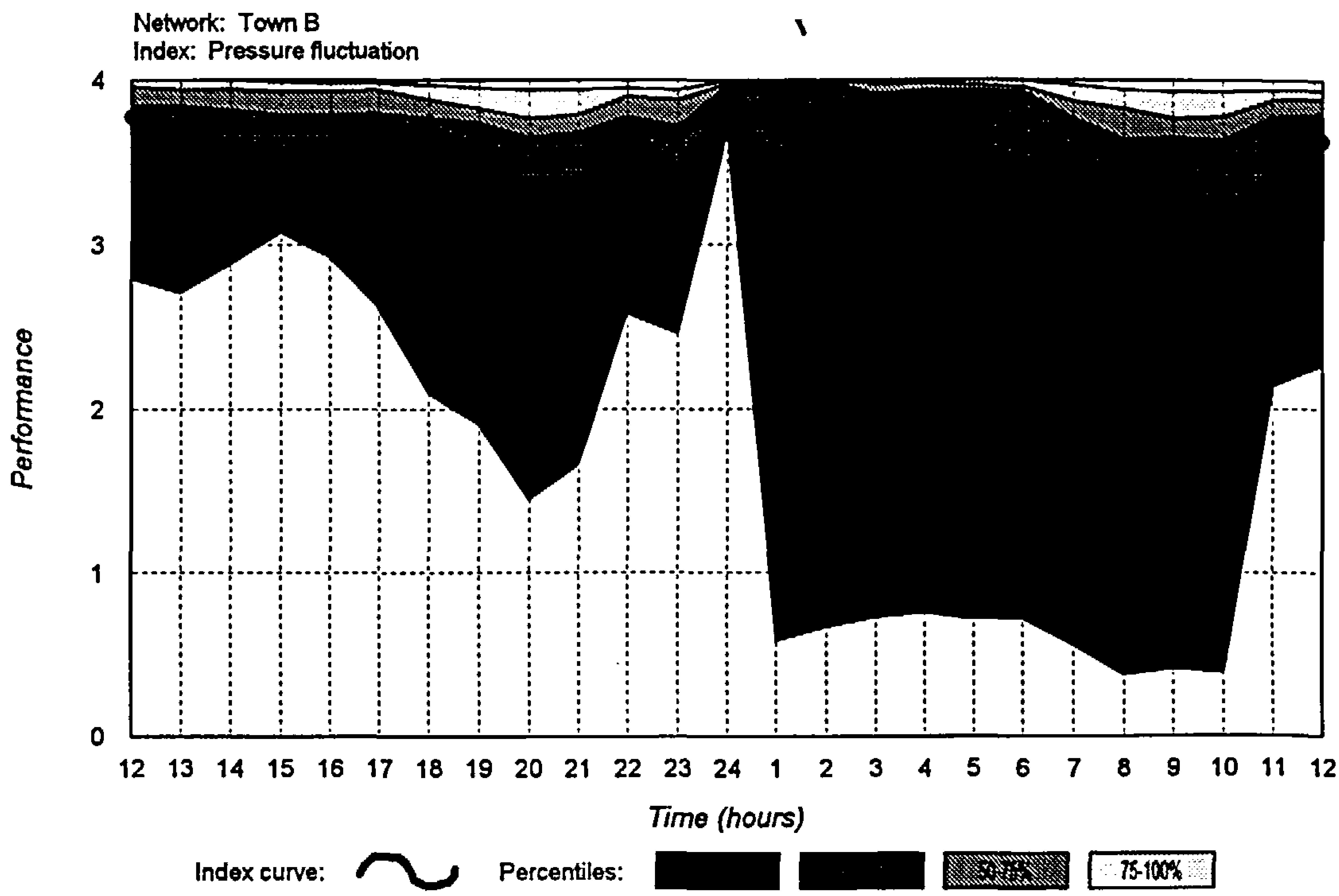

Fig.4.12 - Town B extended-period performance simulation for pressure fluctuation

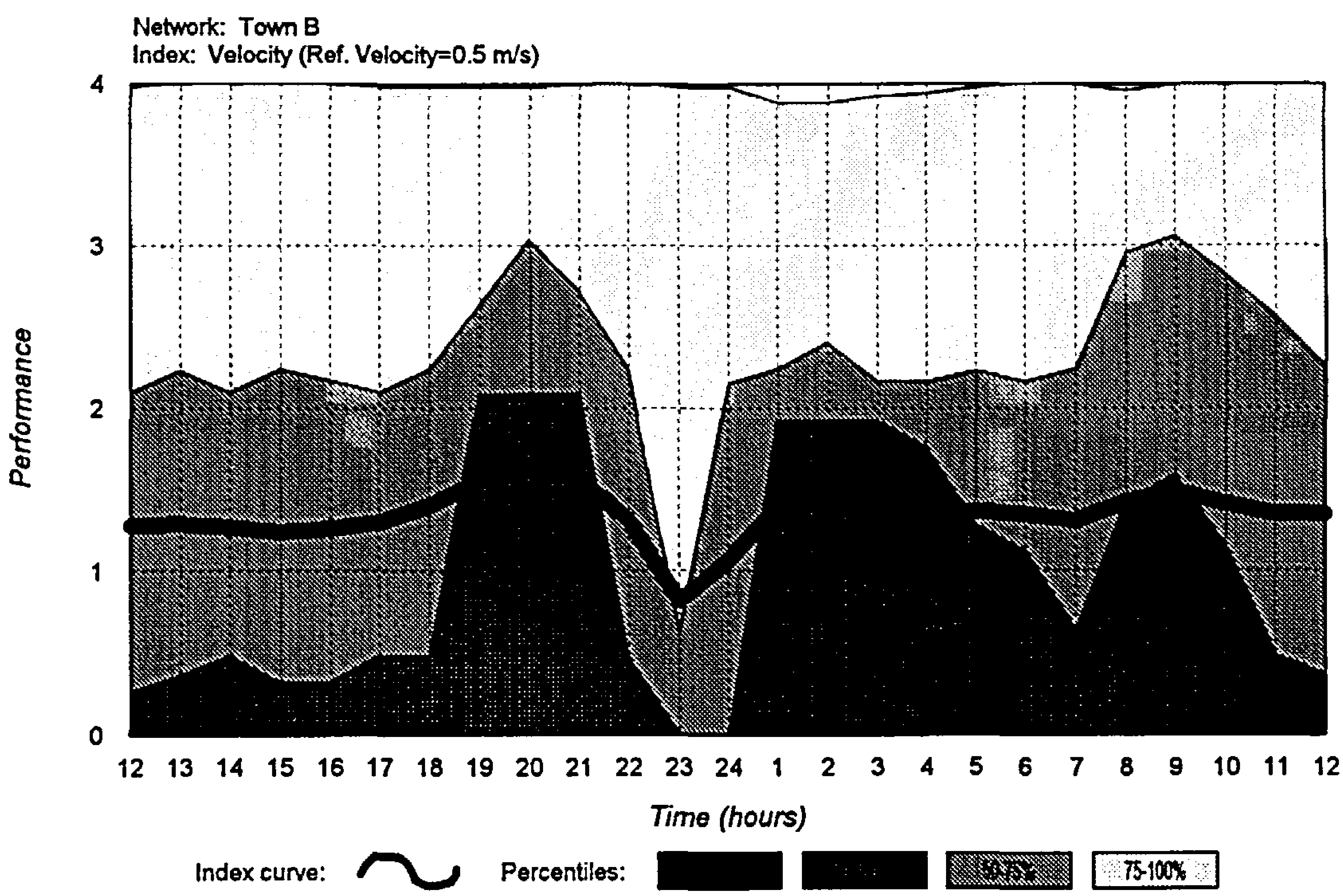

Fig.4.13 -Town B extended-period performance simulation for velocity

Both pressure-related indices are close to the optimum in most of the domain, and with low dispersions. There are localised problems that cause an occasional fall in the lower percentile. The situation as regards flow velocity is more complicated, with great dispersion and probably both too low and too high values.

4.4.4. Town C distribution system

Town C, a large seaside suburb of population 60.000, mainly residential with some industry and a tourist season. The topography is flat, and the distribution system is gravity fed from one reservoir (Fig.4.14). The model comprises 266 nodes and 337 pipes. The network data are given in Appendix A.

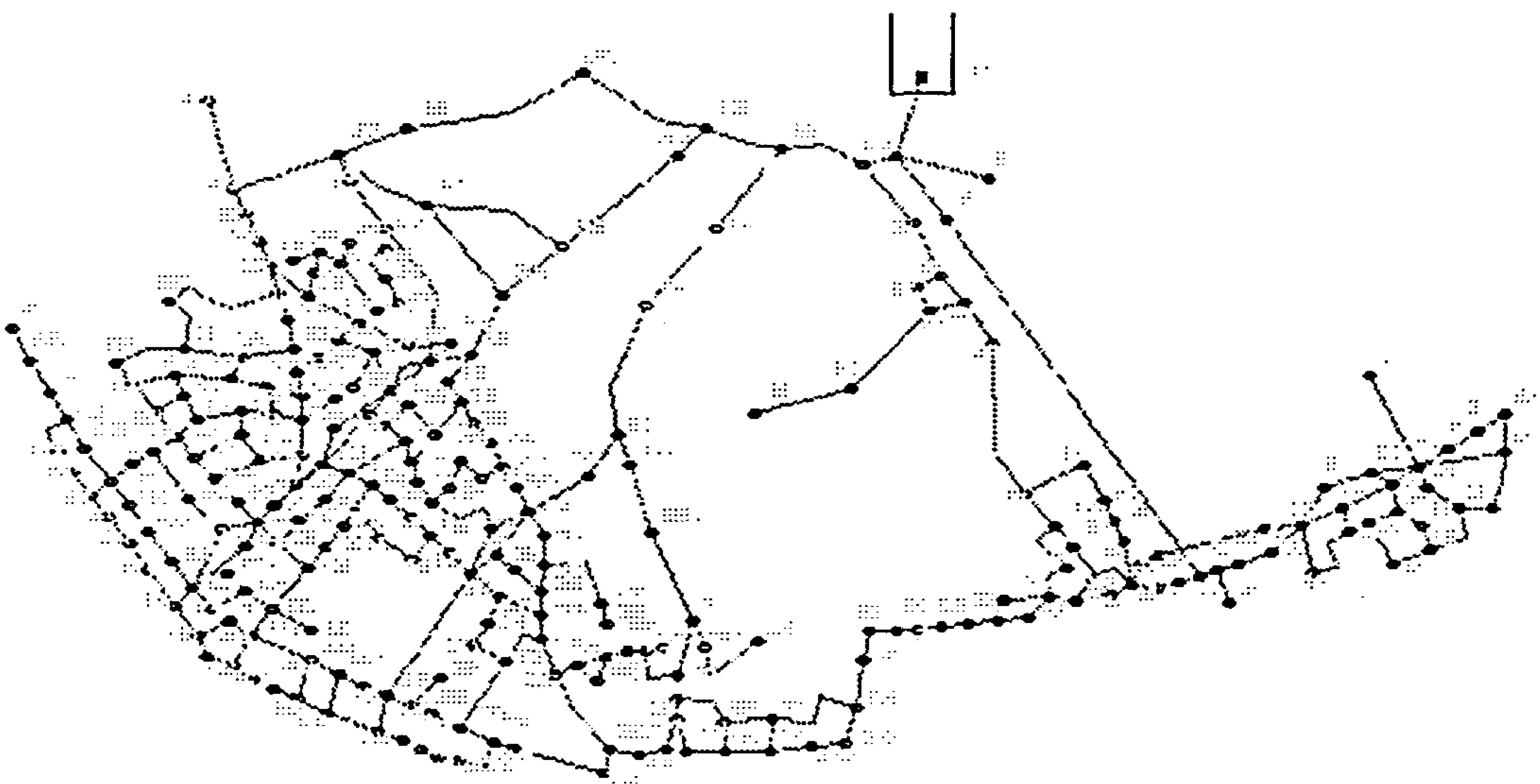

Fig.4.14 - Town C network schematic

System analysis diagrams

The Town C system analysis diagrams (Fig.4.15) display a homogeneous behaviour with good to excellent pressure and fluctuation indices for loads of up to 1.75 times the average. The steep drop above that could be caused by a certain difficulty in specifying the demand factoring without better knowledge of some of the large consumers that the system supplies (see Appendix A). The same increase rate for <u>all</u> the demand types/areas (except leakage), which was used in generating the system diagrams, is possibly unrealistic. If that is not the case, then the system's capacity is seriously limited to the above demand load, which could be cause for concern given the seasonal variations to be expected in a seaside area. The average load depicted here is, however, a summertime demand pattern.

The narrow dispersion bands in the usable domain of the curve show indeed good homogeneity, which is only to be expected from a system with a smooth topography and a relatively favourable layout (the source is not too far from the centre of gravity of demands). The wider 0-25% percentile in the pressure index is due to a small number of nodes, mostly in one particular area (nodes 6, 132, 135, 138, 141, 144, 147). Those nodes apart, the system does reasonably well.

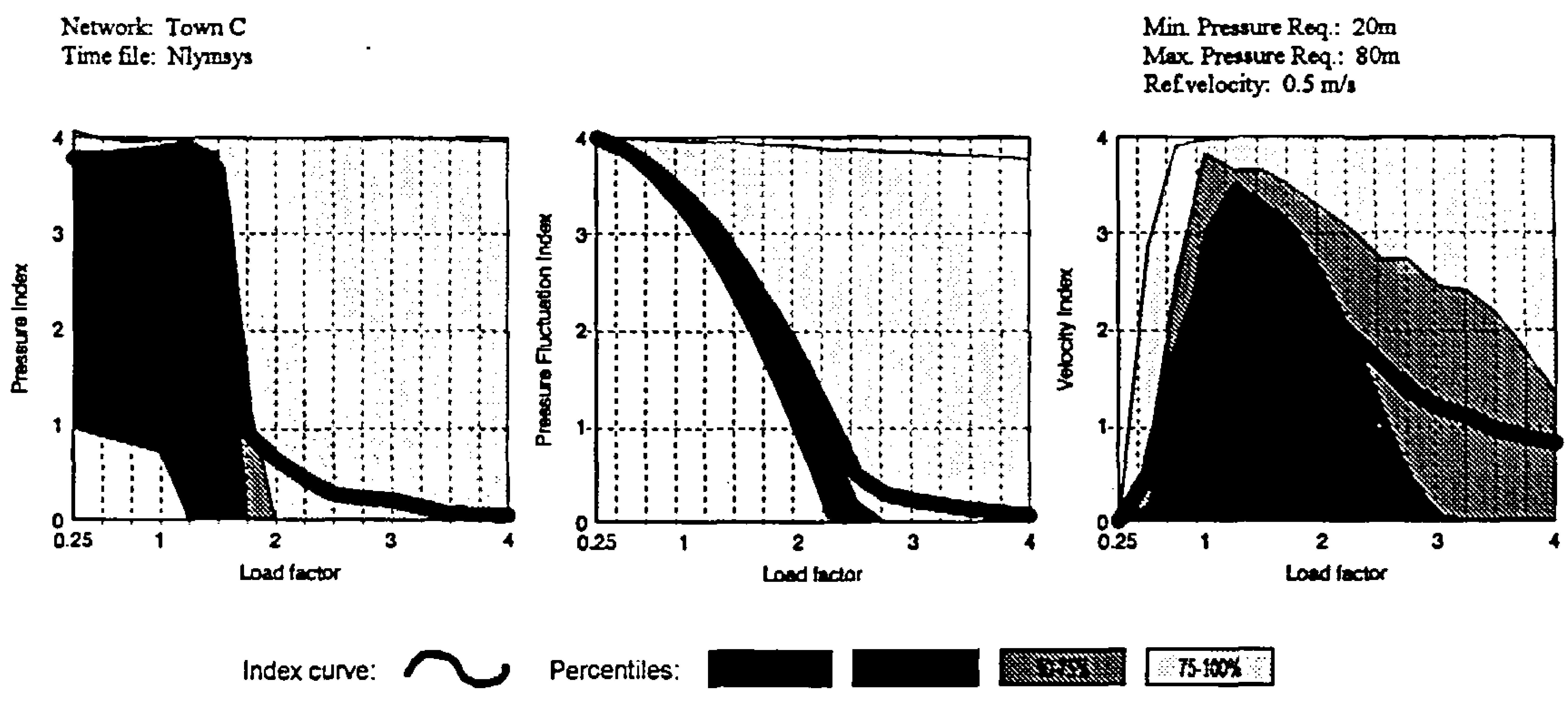

Fig.4.15 - Town C system performance diagrams

The velocity index curve for Town C shows a better behaviour than for the other networks, especially in the average operational load range of 0.75 to 1.75., where dispersion is more contained and all but the lower 25% are above acceptable level.

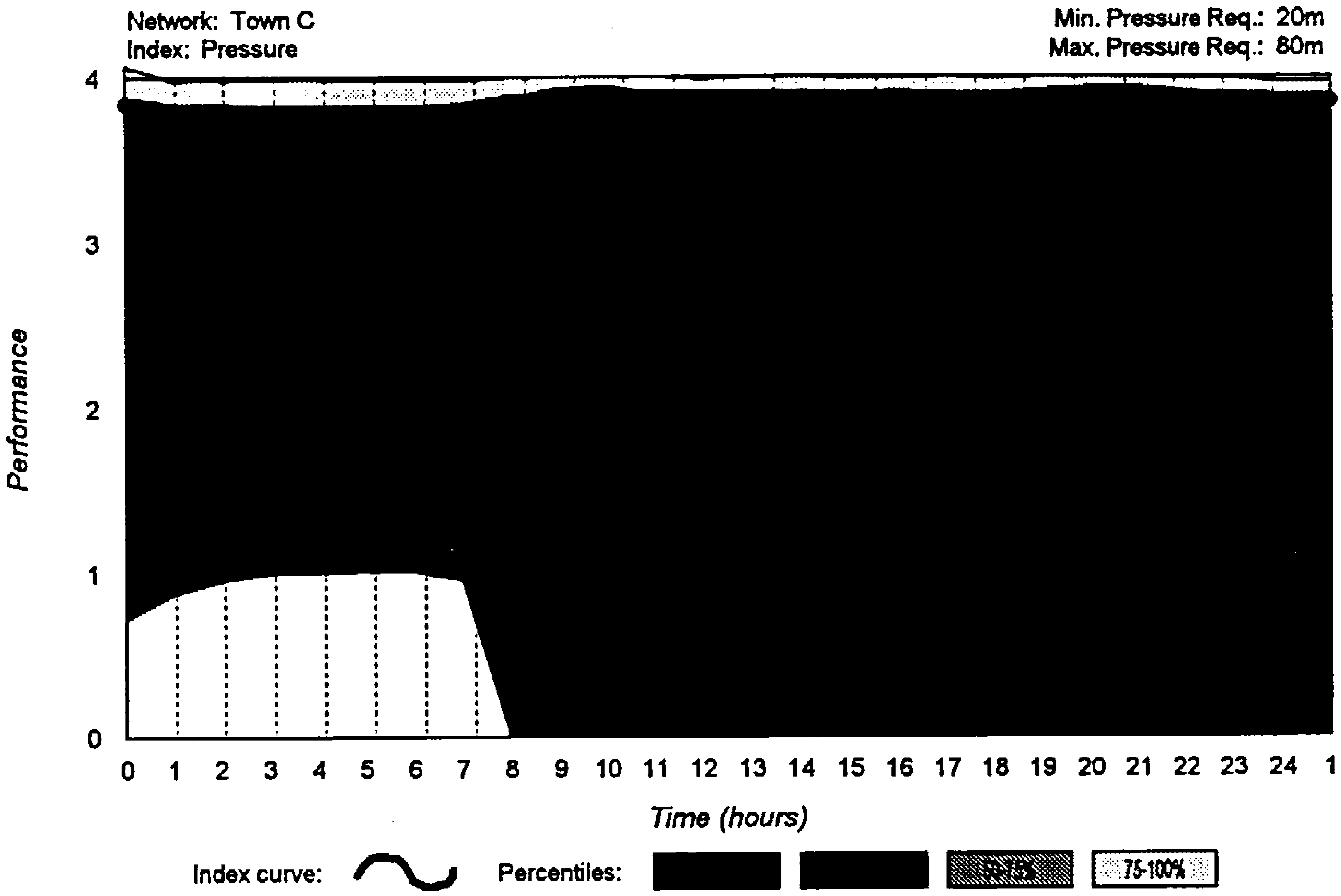

Fig.4.16 -Town C extended-period performance simulation for pressure

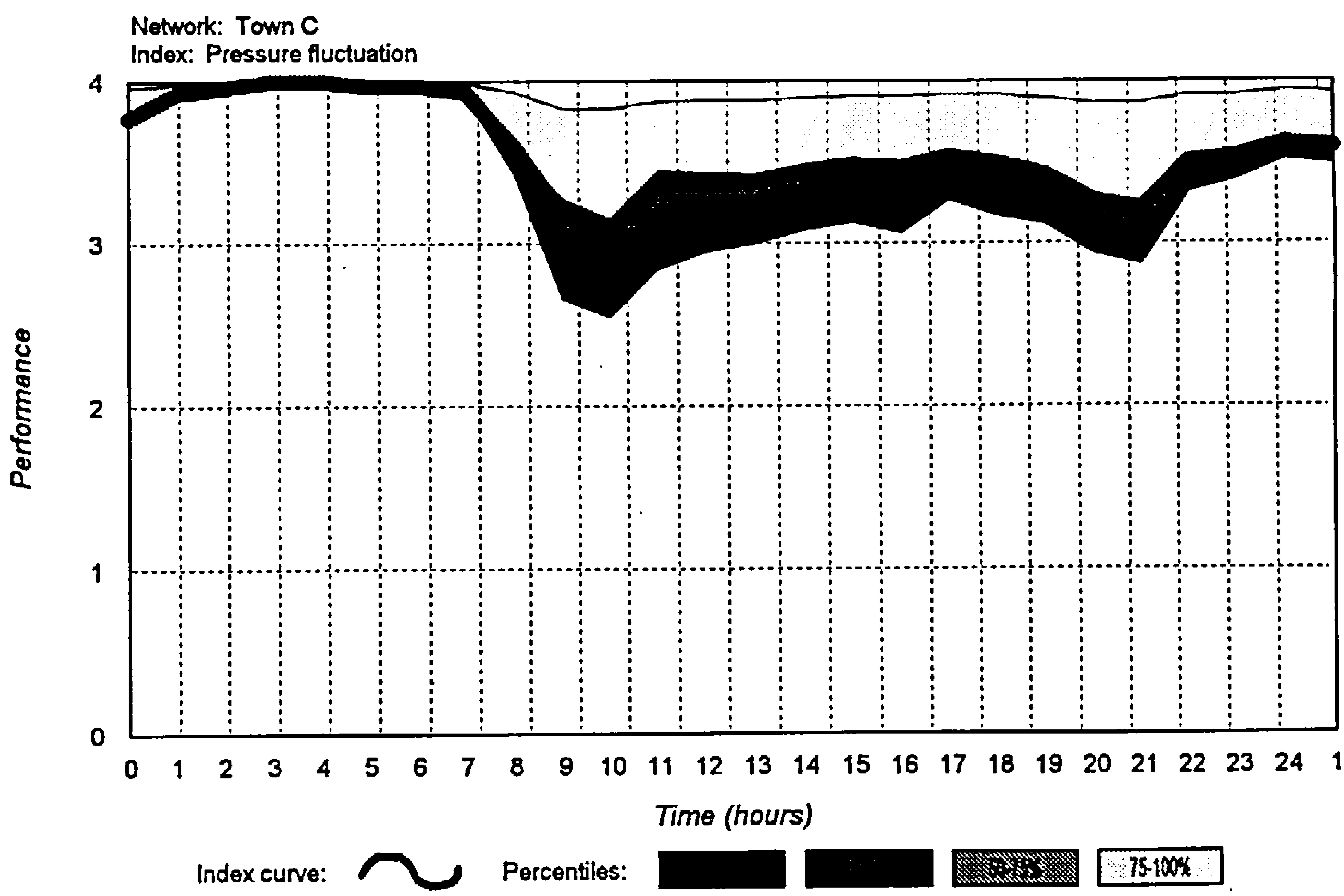

Fig.4.17 -Town C extended-period performance simulation for pressure fluctuation

24 hour simulation diagrams

The diagrams showing the variation of the three performance indices over a 24 hour extended period simulation for the Town C network are shown in Fig.4.16 to 4.18.

As expected for this better known set of demand conditions, the pressure index shows an almost ideal behaviour, which is only spoilt by localised problems mainly around the previously mentioned nodes. The pressure fluctuation is also between good and optimum, and the compactness of the two diagrams shows a very homogeneous behaviour across the network. Although the potential of the network across an extended range may be doubtful, as seen in the system diagrams, the fact is that for the prevailing operating conditions there seems to be no cause for concern.

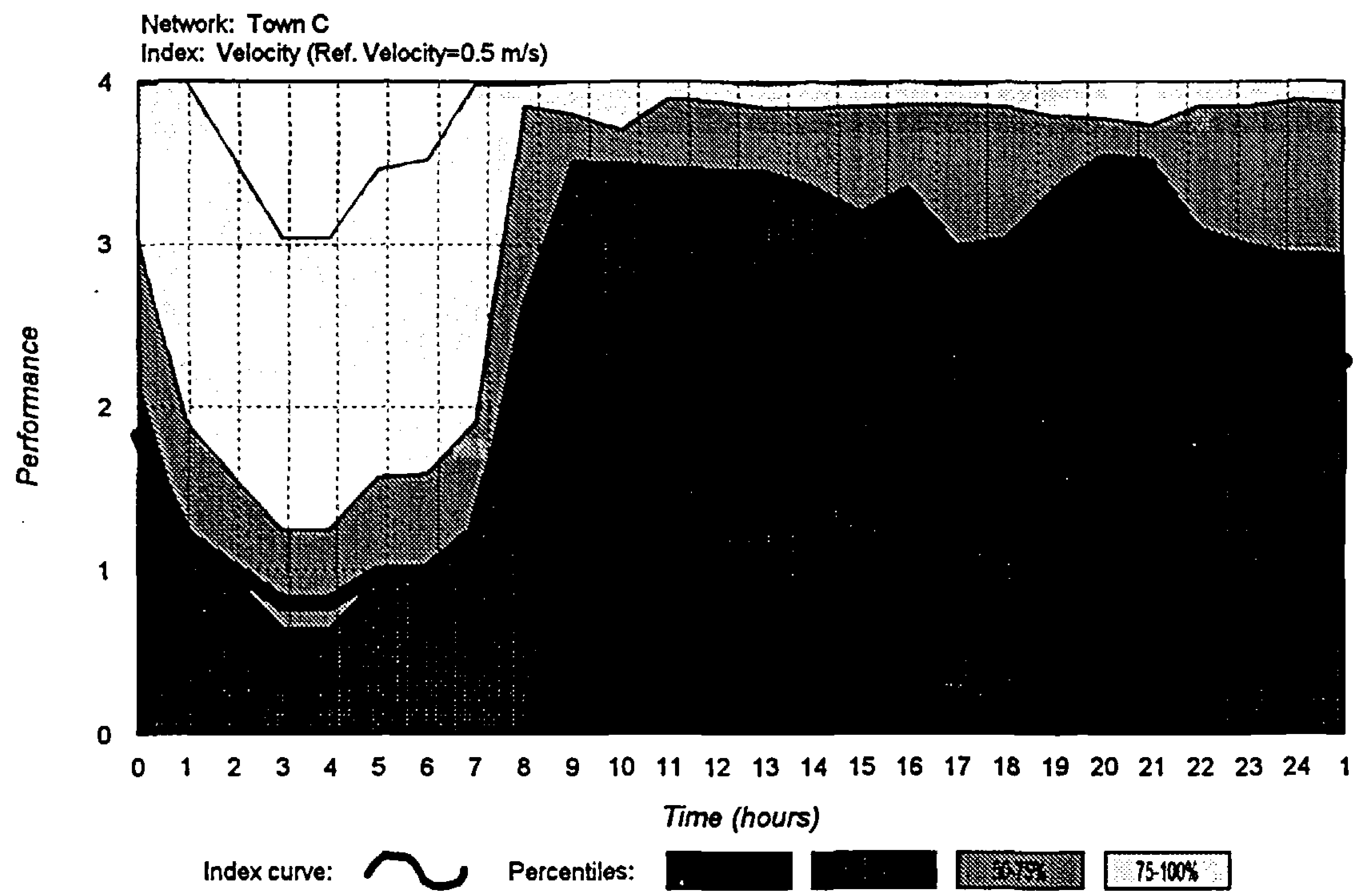

Fig.4.18 - Town C extended-period performance simulation for velocity

The velocity diagram is more scattered, with the night flows well below the reference velocity but the daytime distribution coming closer to an acceptable situation. For the reference velocity used here, 0.5 m/s, this system is over-sized and is generating low velocities during night-time.

4.4.5. Further applications

The following example illustrates some of the uses of the energy-related considerations that were made in section 4.3.4..

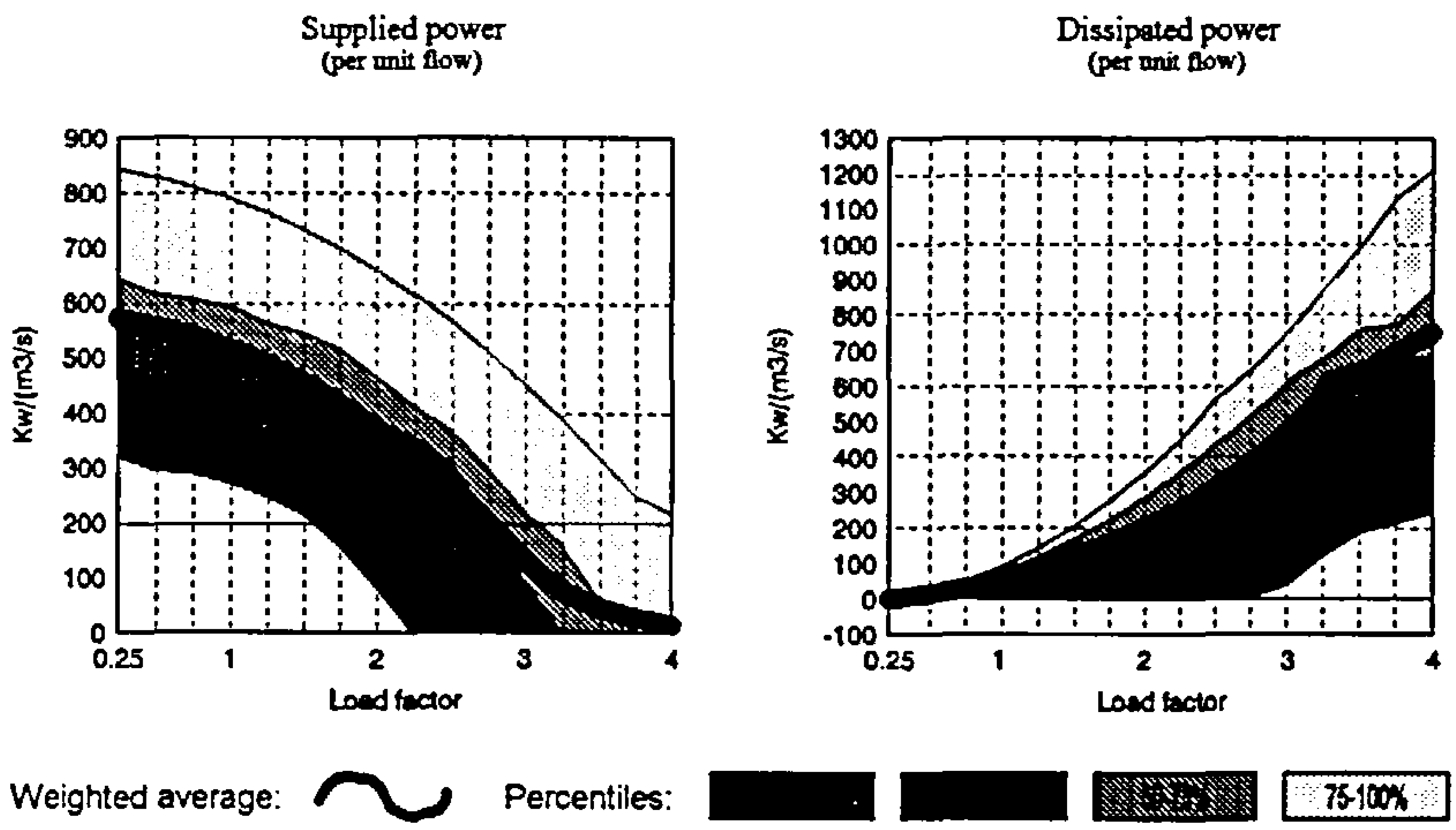

Fig.4.19 - Supplied power and dissipated power

For an example network, Fig.4.19 shows supplied power and dissipated power as given by Eqs.4.19 and 4.15 respectively, but divided by flow to yield power by unit flow. The curves are obtained in the same way as the system graphs for pressure, and the average curve is weighted by the proportion of total demand. In the supplied power graph, the minimum power level, i.e., the power necessary to supply all nodes at the minimum power requirement (20 m in this case) is shown as a solid line at 200 Kw/unit flow. Anything above that line is available surplus power, which the least favoured nodes in the network only enjoy up to a demand load of 1.5, and the network average up to a load of 2.75. In other words, above those values there is insufficient power supplied to the network in order to satisfy the minimum pressure requirement in the nodes concerned.

The dissipated power graph shows an exponential increase with the demand load, which is not unexpected given the relationship between flow and headloss. In fact, much of the insufficiency in power mentioned above stems from the escalating inefficiency of the network as the load increases, which means this system is nearing its efficient capacity for the current demand levels.

Some sensitivity can be gained to the behaviour of the network through simple calculations such as the example that follows. The dissipated power per unit flow for the average demand load is about 50 Kw, while for a peak hour demand load (currently 1.5) is about 120 Kw, or more than double. If that is multiplied by the 1.5 load factor, it shows that the power dissipated by the network at peak hour is 3.6 times higher than at average demand levels. This effect would probably be even more significant in economic terms, as peak hour energy rates are usually higher than the standard tariff.

This type of analysis is a quick means of deciding on the opportunity and cost-effectiveness of investment, for example, for increasing the power efficiency of the system.

4.6. SUMMARY AND CONCLUSIONS

The assessment of hydraulic performance in water supply systems is an increasingly important topic in an industry progressively driven by a need to deliver competent levels of service. On the other hand, the tools and procedures used by designers and engineers are based on more simplistic or fragmented criteria than those implied by the current growing concern over all aspects of the networks' behaviour.

The present chapter develops a set of hydraulic performance measures based on the framework developed in the previous chapter. A brief overview of hydraulic modelling tools in use is given, as a basis upon which the whole analysis is to be built. The selection of what hydraulic state variables to include in a performance evaluation system is then presented and measures concerning pressure, pressure fluctuation, flow velocity and energy consumption are proposed. The corresponding penalty curves and generalising functions are discussed, as well as the suitability of the various measures to the framework proposed and to the objectives of the work.

Illustrative examples are given and the use and potential of the methodology explored. The results presented for some realistic case study clearly show that it is possible to manipulate the information produced by current network analysis to capture a better understanding on some aspects of the system.

CHAPTER 5

WATER QUALITY PERFORMANCE OF WATER DISTRIBUTION SYSTEMS

5.1. INTRODUCTION

The second performance area explored in this work is concerned with the quality of water distributed. As with other previously explored performance areas, water supply and distribution companies are required to meet service standards relating to the potability and aesthetic aspects of the water delivered to their customers. Potable water must meet restrictions on its microbiological contents, as well as on the concentration values of chemical, biochemical and physical substances carried with it. Additionally, the latter may influence certain characteristics of the water that affect its appearance, odour or taste in a negative way and must therefore be controlled to ensure consumer satisfaction.

Despite its chemical, microbiological and aesthetic quality being adjusted at the treatment plant to ensure that it is both safe and palatable to the consumer, the fact is that before it reaches its final destination the water must travel through a distribution system. This may take more or less time, but is almost sure to have an effect on the quality of the water delivered, due to the various transport, mixing and transformation processes that are induced by the complexity of the network along the period of time between production and consumption. Water quality will vary in space and time across the network, often with deterioration of its aesthetic properties — odour, taste, colour, turbidity — and of its chemical, physical and microbiological contents, bringing about the danger of contamination.

Not unlike the hydraulic analysis of a network, water quality analysis of supply and distribution systems can basically be performed in two ways: by direct network sampling and through mathematical modelling. The first alternative is the means of choice to verify the compliance with the applicable standards, and its use is widespread mainly for monitoring purposes. Although undoubtedly irreplaceable and essential in that role, it is nevertheless an expensive and limited process, not very practical for the exhaustive knowledge of the system's behaviour which is often necessary as a basis for much of the planning, design and management of water utilities. However, just as hydraulic network analysis can give us excellent estimates of the network's hydraulic variables, it is possible to obtain a sufficiently accurate picture of the behaviour of certain categories of water quality parameters by means of mathematical modelling, supported by limited monitoring for validation purposes.

Water quality mathematical models are gaining increased recognition as an effective tool for predicting the quality variations in space and time occurring in the supply and distribution process. Water quality models in water supply and distribution can be divided into two major groups: those concerned with modelling the processes that influence the movement and transformation of chemical, biochemical and physical properties of the water, and those that aim to translate the processes governing microbiological life in the networks.

The first group makes use of equations describing the advection, mixing and transformation of conservative and non-conservative substances and parameters, such as turbidity, chlorine residuals or trihalomethanes. These are well known processes: advection and mixing are purely physical and described by reliable and accurate formulations — basically the hydraulic equations of network analysis — and the transformation phenomena (decay, growth or reaction) are adequately studied for many chemical and biochemical substances. As a result, a variety of models has been developed with generally useful results.

In contrast, the second group is rather more difficult to develop and calibrate, given the additional degrees of freedom and the number of extra factors introduced by the biological processes ruling the presence of bacteria and other forms of life. Here not only the advection and mixing processes are involved, but the life patterns and interaction with the environment — as for example with the biofilm growth on pipe walls, which is washed away when it grows beyond certain proportions or when flow velocity increases abruptly — make it considerably more complicated to develop adequate models (Clark *et al.*, 1994, provide a good introduction to this domain). There is equally an increased and much more stringent dependency on practical experiments and field calibration, as compared with the (already in itself rather high) validation requirements of the first group of models. It should therefore be noted that the present work will concern only the modelling domain of the aforementioned first group. The interested reader may find informative references to the modelling of microbiological processes in water supply and distribution networks, and to the sampling strategies that are widely used in that domain, in the aforementioned work by Clark *et al.* (1994) and also in Haas *et al.* (1990), Maul *et al.* (1989), El-Shaarawi *et al.* (1985, 1981) and Means and Olson (1981).

In parallel with the previous chapter where hydraulic performance measures were developed, this chapter proposes to apply the standardised performance assessment framework to the field of chemical, biochemical and physical water quality in distribution networks. As seen before, the performance evaluation framework establishes three types of entities for each network property or behavioural aspect it analyses: (i) A relevant state variable, that is, the quantity which translates the said property at the *network element* level, from the point of view taken into consideration; (ii) a penalty function, mapping the values of the state variable against a scale of index values; and (iii) a generalising function, used for extending the element-level calculation across the network, producing zonal or network-wide values. The indices are intended to have both local and network-wide meaning.

The first step is then to select relevant variables and obtain their values through appropriate modelling. In contrast to the hydraulic performance measures, for which there are well known and widely available models, water quality modelling is a less developed domain. Not only are the commercial or public-domain models less available than their hydraulic counterparts, but the techniques documented for those or published in the literature still lend themselves in most cases to some improvement. The present work therefore includes the complete development of a water quality model.

After introducing the subject of water quality modelling in supply and distribution networks, the present chapter reviews the available methodologies, describes the general formulation governing the processes to be modelled and highlights the solving paths.

The development of a new model, specific to the present work, is then presented in detail. A complete description is made of the numerical algorithm for solving the dynamic water advection and mixing formulation, both in flow through the pipeline system and through storage and other devices. The suitability of the algorithm is discussed with emphasis on numerical accuracy, and different solutions are presented for a numerical diffusion problem arising from computing limitations. The transformation component is then introduced, with discussion of the appropriate models and their extension from the modelling of constituent concentration to the calculation of travel time and source contribution .

The computer implementation of the method is presented, and application examples discussed in order to highlight the different aspects of the model.

The second and third stages in the performance assessment process are the development of penalty functions and the corresponding generalising functions for some of the most typical water quality problems faced by distribution system managers. These are discussed with the help of illustrative examples. The suitability of the performance framework for water quality is analysed and concluded upon.

5.2. WATER QUALITY MODELLING IN DISTRIBUTION NETWORKS

5.2.1. Water quality in distribution networks

The water supplied by distribution networks is destined to be used for human consumption: for drinking, cooking and other domestic uses such as washing and cleaning. It is also used for commercial activities and for those industries without particular quality requirements, as well as for public use such as fire-fighting, street cleaning and the watering of green areas. Even though it stands only for a small fraction (about 1 to 2%) of all the water supplied by an urban utility, it is normally the drinking and cooking water that presents the highest quality requirements. Water must be pleasant in appearance, taste and odour, and must not be harmful to the consumer's health.

Drinking water and public health are very closely related. The human body needs not just the vital liquid but also the mineral salts contained in a good quality water for an adequate balance. As a hygiene product, water helps to prevent the development of many dangerous micro-organisms. On the other hand, water can also be the vehicle for certain maladies, appropriately named waterborne diseases, such as cholera, hepatitis or dysentery, which are a very real threat even in developed countries (Clark *et al.*, 1991, report on a water-related outbreak of diarrhoea in the U.S.A. the previous year, afflicting 240 people and killing 4). Water can equally contain certain chemicals, mainly originated in groundwater by fertiliser residues or industrial waste, which can be harmful if ingested for long periods of time, or even plain toxic.

Other than the consumer-specific requirements, the water must also be kind to the distribution system itself, avoiding corrosion and incrustation.

Water quality problems are an increasing concern of water distribution managers, in the face of ever tightening regulations and control, generalised public awareness and improving knowledge of the various phenomena involved. The water supplied to the consumer by a distribution water utility contains a variety of substances and components:

(i) Substances present at source

Water has the capacity to dissolve, carry in suspension or combine itself with a large number of substances. It is also an ideal habitat for an immense variety of living organisms. Rainwater absorbs or reacts with certain elements in the air, as in the case of carbon dioxide which produces carbonic acid. When the same water becomes groundwater, the carbon dioxide helps to dissolve various elements such as iron, manganese, calcium and magnesium, carrying them in solution. Surface water will carry a great variety of organic and inorganic materials in suspension. Wastewater and other residues, dumped in rivers and lakes with insufficient treatment, also contribute to introduce contaminants and other harmful substances.

There are over 2000 different parameters already identified as potentially present in the water at source. In a broad simplification, the following are the main types of source water:

- acid (excess carbon dioxide), mainly aggressive to the distribution system components;
- hard and associated with lime, the potential cause of incrustations in pipes and reportedly related to high frequencies of kidney problems and to low frequencies of heart disease when occurring in a high degree— average lime waters do not seem to have any effect on public health;

- with iron and manganese, causing turbidity, incrustation in iron pipes and equipment, and staining clothes and sinks;
- coloured or turbid, which may also be cause taste and odour, and may be harmful to public health depending on the dissolved or suspended substances;
- bacteriologically contaminated, a public health risk through the presence of micro-organisms which can cause disease; and
- chemically contaminated, or containing toxic substances dangerous to human life.

ii) substances introduced as part of the treatment processes, such as chlorine disinfectant

The objective of treating drinking water is to make it potable, palatable and as kind as possible to the distribution system. That is carried out by correcting any inappropriate characteristics and by introducing elements of further protection that will help guarantee its potability along the path to the consumer's tap.

The most essential treatment process is disinfection, which purports to eliminate any pathogenic micro-organisms either present at source or potentially present further along the network. Chlorination is the most common disinfection procedure, with a residual action that protects the water even after its application. Chlorine may be added to the water in various forms and the dosage depends on the type of water as well as on the contact time of the agent – sodium hypochlorite, chlorine gas, etc. – with the water. Apart from chlorine residuals, this type of disinfection can also leave less desirable by-products in the water, such as trihalomethanes (THM's), and it is not always easy to strike a balance between those and the correct levels of disinfection.

Other treatment processes help improve different aspects, such as: the correction of acidity through contact with calcium carbonate or by adding lime or sodium hydroxide; the correction of hardness by precipitation or ionic permutation; the removal of iron and manganese through precipitation induced by aeration or chemical oxidation, plus filtration; the removal of turbidity through coagulation-flocculation and filtration; etc..

iii) substances and alterations introduced by the distribution network

The variability of water quality within the network is the key issue, as it becomes increasingly clear that the distribution system has a definite effect on it. Water leaving the treatment plant as a reasonably known quantity, in terms of its quality parameters, may quickly become an unknown as it flows through the network to the end users. A water distribution system is indeed a rather large and complex "living" system, where physical, chemical and biological processes combine to introduce mostly undesirable changes to the composition of the water flowing through it.

The following are some of the factors that may have an influence on water quality between treatment and the consumer (Clark, 1993a, Grayman et al, 1988):

- Chemical and micro-biological quality of the water at the source and after treatment;
- adequacy and efficiency of the treatment processes employed;
- age, type, layout and condition of the distribution system;
- condition of storage facilities;
- in case of multiple water sources, mixing of waters with different properties;
- consumption patterns, particularly those that escape the norm;
- hydraulic adequacy of the distribution, which determines the travel times throughout the network and residence times in storage.

Changes in water quality happen both within the water itself and through its contact with the boundary walls of pipes, tanks and remaining components of the system. Examples of some of the processes that cause them are: chemical precipitation and flocculation of certain substances; the decay of disinfection agents, interacting with organic and inorganic compounds to produce changes in appearance, odour or taste, or bacterial growth; extraneous interference, such as resulting from construction works, repairs, pipe breakage, etc., introducing external materials and particles; sudden abnormal flows or mains flushing unsettling existing deposits and biofilm; unfavourable pressures drawing in raw or contaminated groundwater through leaks, especially if neighbouring sewers are at higher elevations; stagnation in low-velocity mains, dead-ends and storage tanks; internal corrosion of pipes and general reaction between the water and network materials; etc..

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An effective quality control of drinking water can only be guaranteed by combining a wide range of measures such as:

- The prevention of contamination of water resources;
- adequate protection of intake points;
- appropriate treatment processes relative to the type of water;
- efficient maintenance of the supply and distribution system;
- regular, systematic quality monitoring of the water distributed to the consumers; and
- quick reaction capabilities when quality problems are detected.

Sampling is the primary means of monitoring and assessing the quality of the water in a distribution network. Indeed most of the legislation requires sampling programmes that periodically cover selected and random points throughout the entire system, analysing for physical, chemical and microbiological parameters from source to end user. There is increased emphasis on sampling at the tap to test compliance rather than sampling at the sources or treatment works. As mentioned before, however, it is an expensive, constraining and less than practical process for the exhaustive knowledge of the system's behaviour which is often necessary as a basis for much of the planning, design and technical management activities of water utilities.

The present section focuses on water quality models, one of the primary tools available to the water engineer and manager for the control of water quality in distribution networks and as an invaluable aid to the planning and interpretation of sampling programs. The subject of water quality models is presented in detail in the next sections, including the development of a specific model and illustrative examples.

5.2.2. Water quality modelling

Computer-based mathematical models for assessing the movement and change of waterborne substances in distribution systems emerged during the 1980's, following the widespread adoption of computer models for hydraulic network analysis in the previous decade. They are generally divided into steady-state water quality models and dynamic water quality models. These models normally require the previous use of network analysis (correspondingly, steady-state or extended period simulation) for determining the set of hydraulic variables describing the behaviour of the network, prior to calculating the movement and evolution of substances in the water. Some models can only trace conservative substances, that is, substances which do not experience any change in mass with time or through interaction with other substances, while others will also model decay or growth along the network.

Not unlike the network analysis models upon which they are based, water quality models can also be divided into simulation models, which are mainly used to describe the processes occurring in the system, and optimisation models, which go a longer way in attempting to achieve a best solution to a particular design or operational problem.

Water quality models are used to trace the movement of contaminants or other elements in the water, to assess their decay or growth along their paths, to calculate the age of the water in the pipes, or to determine the origin and destination of water at any point of the network.

The following fields are frequently mentioned as those where water quality simulation models offer potential for direct application (Clark, 1993, Tansley and Brammer, 1993):

- prediction of water quality degradation problems;
- prediction of contaminant dissemination and design of flushing strategies to face possible pollution incidents;
- establishing the relative contributions of different sources at any point in the network, when problems caused by blending of waters occur;
- design of water quality sampling programs;
- optimisation of the disinfection process, including investigation of best location for booster chlorination within the network;
- assessment of the effects of repairs or rehabilitation;
- evaluation of operational network control strategies; and
- storage planning and design.

However, depending on how critical water quality constraints are in specific situations, water quality models can conceivably drive the entire process of planning and designing a complete water supply and distribution system. Furthermore, with the increasing emphasis placed on the tight control and enforcement of water quality regulations, this is seen more and more as a major concern in many of the areas of technical management of a water utility.

5.2.3. Problem formulation

The movement and transformation of a waterborne substance in a water distribution network comprises three major processes. Two are mainly due to the underlying flow of water: advection along the pipelines and mixing at pipeline junctions. The third, affecting non-conservative elements, is the substance's own transformation process: reaction within itself, reaction with other substances present in the water, and reaction with its boundary material, the pipe or storage container wall. In all three cases the result may be growth or decay of the substance, and/or transformation into a different substance.

Another effect that may be taken into account is the longitudinal mixing along the direction of flow. However, this is a process that is usually considered to be small relative to the bulk advection, and is therefore normally neglected (Grayman *et al.*, 1988, Liou and Kroon, 1987). On the other hand, cross-sectional homogeneity is assumed, given the range of pipe sizes normally employed in distribution networks.

The advection process is fundamentally modelled by the equations of hydraulic network analysis (see Chapter 4), in as much as these provide a complete description of the network's hydraulic behaviour. That is translated by means of its state variables, such as pressure head, headloss gradient, flowrate and flow velocity. The last two provide the necessary information to model the advection of any waterborne substance, although exactly how is seen subsequently.

It must be noted, however, that the use of network analysis results means that all its assumptions and simplifications, as described in the previous Chapter, are also constraints of the present modelling methodologies.

The mixing process occurring at pipe junction nodes is modelled under the assumption that complete mixing takes place. That is, given a set of pipes contributing to a generic node and carrying equal or diverse concentrations of a particular constituent, the concentration of that substance on all pipes leaving the node will be the same, fully mixed blend[1].

By further assuming mass conservation for the substance across the junction node, the concentration for all outgoing pipes can be expressed in terms of the incoming streams' concentrations. For a generic node i, with U^i upstream nodes and D^i downstream nodes:

$$C_{ij} = \frac{\sum_{k=0}^{U^i} q_{ki} C_{ki}}{\sum_{k=0}^{U^i} q_{ki}} ; \forall j \in D^i \tag{5.1}$$

where:

q_{ij} - flow from i to j

C_{ij} - concentration of flow leaving node i for node j

q_{0i} - supply flow introduced at node i from any external source

C_{0i} - concentration of supply flow at node i

In short, equation 5.1 states that the average concentration of the particular constituent at node i, and on all pipes leaving i, equals the total mass of constituent entering the node from all its contributing pipes and influent sources, divided by the total flow through the node.

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1 This is a widely accepted assumption. It is reasonable enough for substances which are present in solution form or parameters thereof. In the case of substances in suspension, such as those causing turbidity, the assumption should be taken with appropriate caution since it may not always hold.

The transformation process, which results in changes in the concentration of a substance as it is carried along with the flow, is taken into account in the following one-dimensional mass conservation differential equation, which in essence translates the advection process mentioned before. For the generic link connecting node i to node j:

$$\frac{\partial C_{ij}}{\partial t} = V_{ij} \frac{\partial C_{ij}}{\partial x_{ij}} + RF(C_{ij}) \tag{5.2}$$

where:

t - time

x_{ij} - distance along pipeline connecting nodes i and j

V_{ij} - velocity of flow from i to j

$RF(C_{ij})$ - reaction rate function of substance carried in flow from i to j

In fact, C_{ij} in the above equation is a function of both distance and time:

$$C_{ij} = C_{ij}(x_{ij}, t) \tag{5.3}$$

The problem therefore consists of sequentially solving a differential equation at each pipe, Eq.(5.2), for which the initial condition (t=0) is known, subject to a boundary condition at x_{ij}=0 given by Eq.(5.1) thus re-written:

$$C_{ij}(0,t) = \frac{\sum_k q_{ki} C_{ki}(L_{ki}, t) + q_{0i} C_{0i}}{\sum_k q_{ki} + q_{0i}}, \forall k \in U, \forall j \in D \tag{5.4}$$

where L_{ki} is the length of pipeline connecting nodes k and i.

5.2.4. Steady-state water quality models

Steady state models of the propagation of waterborne substances in distribution systems generally make use of the laws of mass conservation to achieve the equilibrium concentration distribution which would ultimately occur when the network's hydraulic equilibrium is reached.

One of the earliest proposals for water quality modelling available in the literature, Males *et al.* (1985), presented one such algorithm for modelling mixing problems in water distribution systems under steady-state conditions. The algorithm is based on known flows throughout the network, obtained through hydraulic simulation as described previously. Writing the mass balance at the nodes in terms similar to equation 5.1, assuming complete mixing takes place, yields a set of linear equations for constituent concentration, which can be solved by standard methods. The results are the equilibrium distribution of the constituent throughout the network under steady-state conditions and therefore do not consider any variations with time.

A number of steady-state techniques have been proposed, including Chun and Selznick (1985), Clark *et al.* (1988), Wood and Ormsbee (1988), Ostfeld and Shamir (1992), and Boulos *et al.* (1993).

Steady-state models are useful tools for general study and sensitivity analysis of a network. This is generally accepted at the hydraulic analysis stage, but it is widely recognised nowadays that even for networks with nearly constant operational conditions, the time required for a given substance to spread out and reach some sort of equilibrium distribution will not be achieved before variations occur in the demand patterns throughout the day.

Dynamic water quality models are deemed more suitable for studying those processes as they can take into account the varying hydraulic scenarios, offering a better representation of the time-dependent interaction between hydraulics and waterborne substance spread. For that reason they are regarded nowadays as the tool of choice for water quality studies. The next section introduces such models.

5.2.5. Dynamic water quality models

Dynamic water quality models are based on the full system simulation technique in order to trace the movement and evolution of waterborne substances throughout a distribution network, under time-varying demand, supply and operational conditions. The problem now consists of solving the full set of equations described in 5.2.3., for each hydraulic state.

Grayman *et al.* (1988), and Rossman (1993), both propose numerical solutions to the same formulation, by using a fixed discretisation of the space[2] and time co-ordinates. This technique is known as the discrete volume element method (Fig.5.1).

The procedure solves Eq.(5.2), for the set of flows valid through each interval ΔT of an extended period simulation, by dividing each link of the network into a finite number of discrete volume elements, and propagating the concentration properties along these elements. Each hydraulic time step ΔT is divided into shorter *water quality* time steps δt, each pipe being correspondingly divided into completely mixed elements of length equal to $V \cdot \delta t$, or volume equal to $Q \cdot \delta t$, with V and Q the velocity and flow rate in the pipe, respectively. The water quality time step δt should not be larger than the shortest time of travel through any pipe in the network (i.e., L_{ij}/V_{ij}). For the same δt to be used across the network, the number of elements has to be rounded off to the nearest integer number in any pipe for which the length is not a multiple of $V \cdot \delta t$.

2 In a restricted sense, referring to the longitudinal axis of ezch pipe.

The transformation process equation is then integrated for each of those volume elements throughout each water quality time step δt, at the end of which the mass of constituent contained in the volume element is transferred to the next (downstream) element. At the end of the pipe, when the junction node is reached, Eq.(5.2) is solved successively for the incoming pipes to that node until all have been processed. The resulting concentration is then used as the initial solution and boundary condition for the pipes flowing out of that node.

The sequence of steps is carried out throughout the entire network, during the hydraulic time step ΔT, at the end of which a new set of flows comes into action. The pipes are once again divided and the process is repeated. The concentrations along the pipelines are carried over from the previous hydraulic state, and where the velocities have changed and new pipe elements have been created, an interpolation process is used to transfer the concentration variation to the new discretisation. The methodology is claimed to be rather efficient from a computational point of view.

Although conceptually there is nothing much wrong about the method, it is its implementation that inevitably incurs some serious limitations. Like all numerical solutions of this kind, it suffers from the ill effects of discretisation and numerical diffusion. As Liou and Kroon (1987, 1988), point out, unless the water quality time step δt tends to zero, some degree of

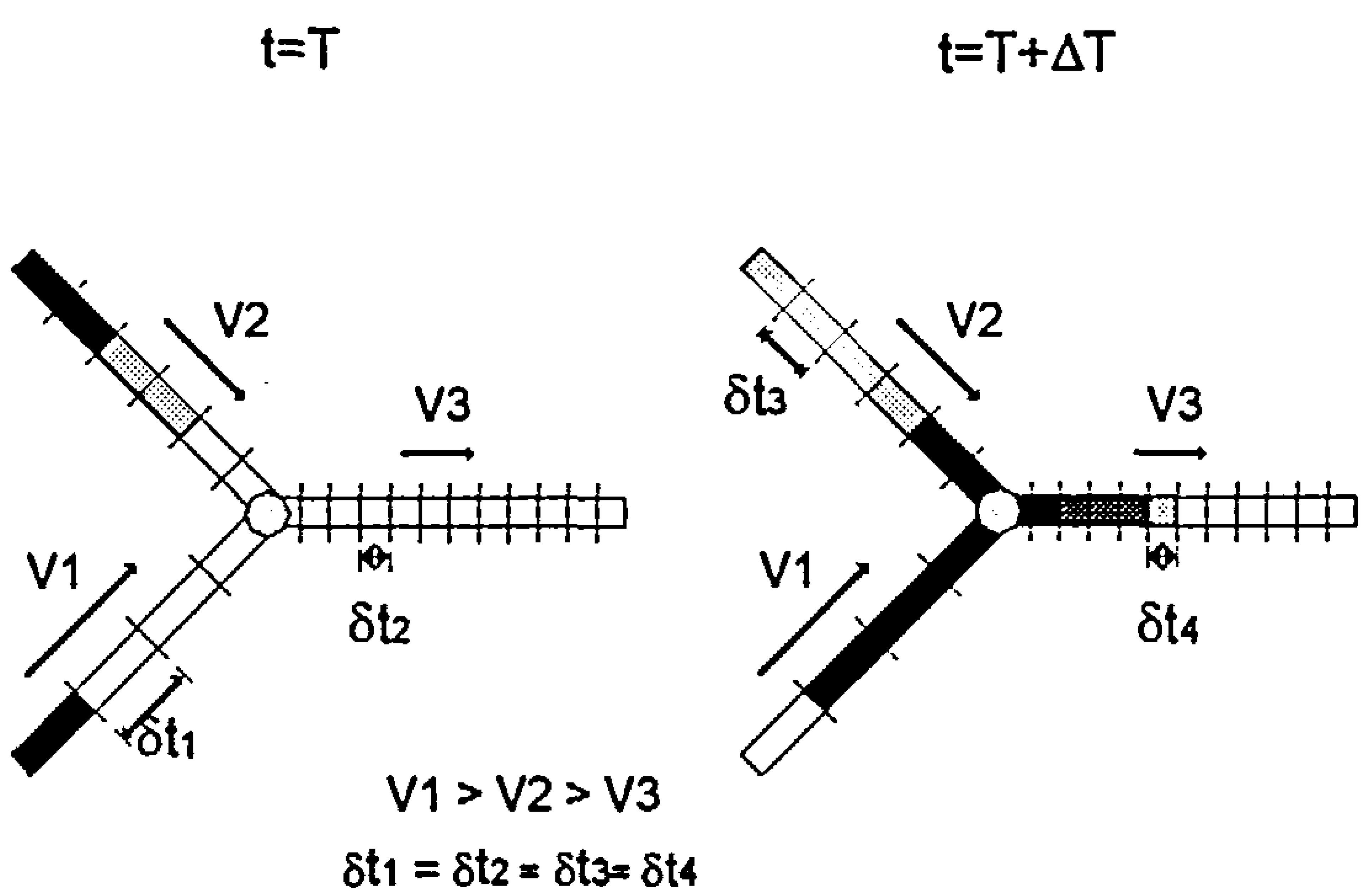


Fig.5.1 - Discrete volume element method

approximation in the movement of the constituent will be induced by the discretisation, and only made worse by the need to round off the number of elements in the pipe. The algorithm is forced to interpolate between different discretisations in successive hydraulic time steps, which will tend to induce further numerical diffusion, especially if flow reversal occurs between two time steps. Finally, the scheme must round off any pipe with a shorter time of travel than δt to zero length, which could be misleading considering δt is typically set by the user, and not specifically calculated to minimise error.

Liou and Kroon (1987), present an alternative to this scheme which does not depend on a

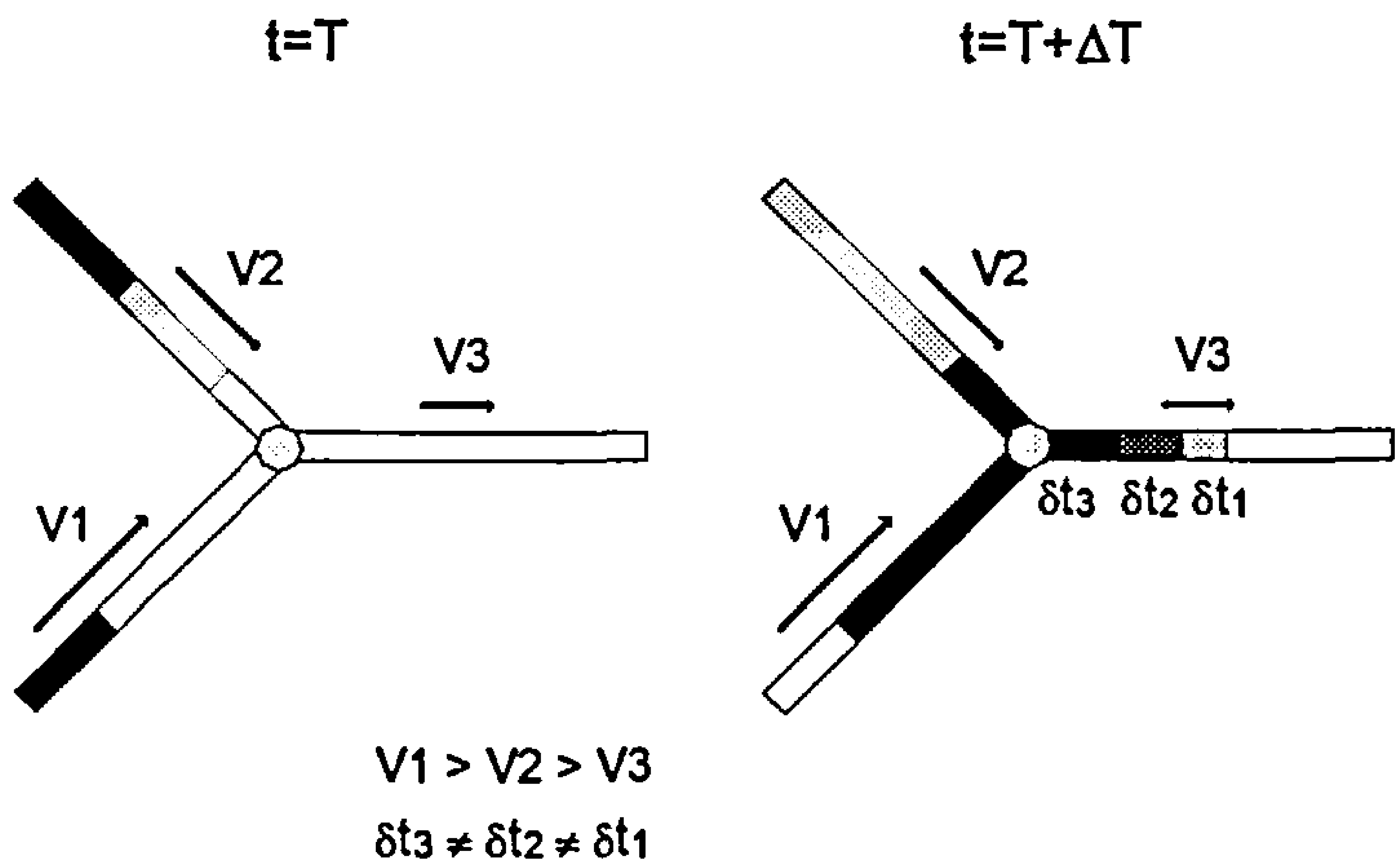


Fig.5.2 - Liou & Kroon's method

fixed δt (Fig.5.2). Instead, they divide the length of each pipeline into a number of variable-length, fully-mixed volume elements so that the concentration variation along it is adequately represented, and then create a number of δt's at the end node to allow for a step-by-step mixing analysis. Those δt's do not have a fixed length, and correspond to all the changes in concentration at the node as a result of the mixing of the different volume elements coming in from all the contributing pipes. The number of volume elements in a pipe is only limited by computational memory constraints — a maximum is defined by the user and an aggregation scheme is deployed to merge the volumes with the smallest differences in concentration.

In spite of the advantages displayed over their steady-state counterparts, existing dynamic models are not free from limitations, sometimes significantly hampering their simulation capabilities. The computing processor and memory requirements generated by those techniques greatly varies, but may reach unfeasible time and memory levels for complex networks, or networks modelled to a great detail (as indeed they should from the water quality point of view, as will be mentioned in further sections). This is especially true, for those models described above, when there are many large pipe lengths with slow velocities. The present work addresses this area through the introduction of a new dynamic water quality model in section 5.3.

5.2.6. Water quality model validation

The success of a mathematical model of a real-life physical phenomenon depends as much on the adequacy of the formulation in itself as on the fit of the parameters it depends on. Calibration is therefore a crucial stage of any attempt at modelling water distribution systems, both for its hydraulic behaviour and for its water quality properties.

The calibration of hydraulic simulation models is probably the most important issue in the validation of any subsequent modelling, including water quality models such as the ones addressed in this chapter. It has been seen that the type of model described here relies totally on the hydraulic solution to yield its description of the advection and mixing processes (and implicitly, the transformation process as well). Therefore, a poorly calibrated hydraulic model will inevitably hinder the capabilities of a water quality model depending on it.

However, given an adequately calibrated hydraulic model, it is possible to a certain extent to validate the further calculations involved in the process of modelling, mixing and transformation of waterborne substances. The principal and best method is to use water quality tracers. Chemicals which either already occur in the water, or are artificially added for the purpose, may be measured in the field and the results used to verify the models. The most common tracer is fluoride, which is approximately conservative, safe for public health[3] and simple to add or reduce in normal daily activity. Its movement can be traced in the system using portable, hand-held analysers. Examples of the use of fluoride as a tracer are described in Clark *et al.* (1991), and Clark *et al.* (1993).

Conservative tracers are used to validate the mixing and advection processes. To test the adequacy of a transformation model (such as Eq.5.2), the specific substance it is intended for should ideally be used. However, such tests are seldom possible in the network without endangering the potability of the water. This is an area where there still is need for laboratory development of models for the specific substances found in water network environments.

An example of a parameter which is better documented than most, especially as regards actual field testing, is chlorine residual. This is a product of disinfection by chlorination at certain points in the network, which is intentionally kept in the water for preventive protection against microbiological contamination. Tansley and Brammer (1993), and Burgess *et al.* (1993), describe instances of calibration of models for chlorine residual.

5.3. DEVELOPING A DYNAMIC WATER QUALITY MODEL

5.3.1. Introduction

In order to achieve the modelling capabilities necessary for exploring the application of the general performance framework presented in this work to the field of water quality, a suitable water quality simulation model was developed. A dynamic formulation would be required in

3 Even though there has been discussion of its suitability in the context of mass-medication strategies.

order to retain the extended-period simulation capabilities of the remaining performance evaluation system.

A model was therefore developed to track the propagation, mixing and transformation of waterborne substances travelling in a water distribution network. The model performs a dynamic simulation of water quality parameters or travel times flowing in a network under steady state or time-varying conditions, and solves the general formulation through a numerically explicit procedure containing some advantages over the methods proposed in the literature.

The model was implemented through a computer program, PERFORMANCE-Q, which can work either stand-alone or as a module of the global performance evaluation system, PERF.

This section presents the main aspects concerning the development of the algorithm and its computer implementation, as well as its application to example networks. Further sections will describe the model's use in performance analysis as part of the global framework.

5.3.2. Proposed methodology

The water quality model proposed in this work solves the dynamic equations previously presented by means of a numerically explicit algorithm which is based on, and driven by, the actual changes in water quality occurring at the sources or anywhere across the network, rather than arbitrary discretisations. It sequentially traces the movement of *volumes* of water of homogenous concentration along the pipes of the network. Such volumes are defined as the plug of flow occurring between two consecutive changes in concentration, either originated at the source nodes by changes in the incoming water, treatment process, etc., or caused by mixing at any node in the network.

A tracking scheme is introduced based on a simple but effective concept of columns of water at the nodes, creating an accurate recursive algorithm which is numerically error-free. The method includes a careful treatment of some numerical weaknesses in previous models, affording a very acceptable accuracy level. The method is simple to translate into computer code, rather efficient in memory requirements and completely free from numerical diffusion except where dictated by computing limitations.

Before formalising the algorithm through an appropriate formulation, it will be helpful to introduce the basic concepts in broad terms.

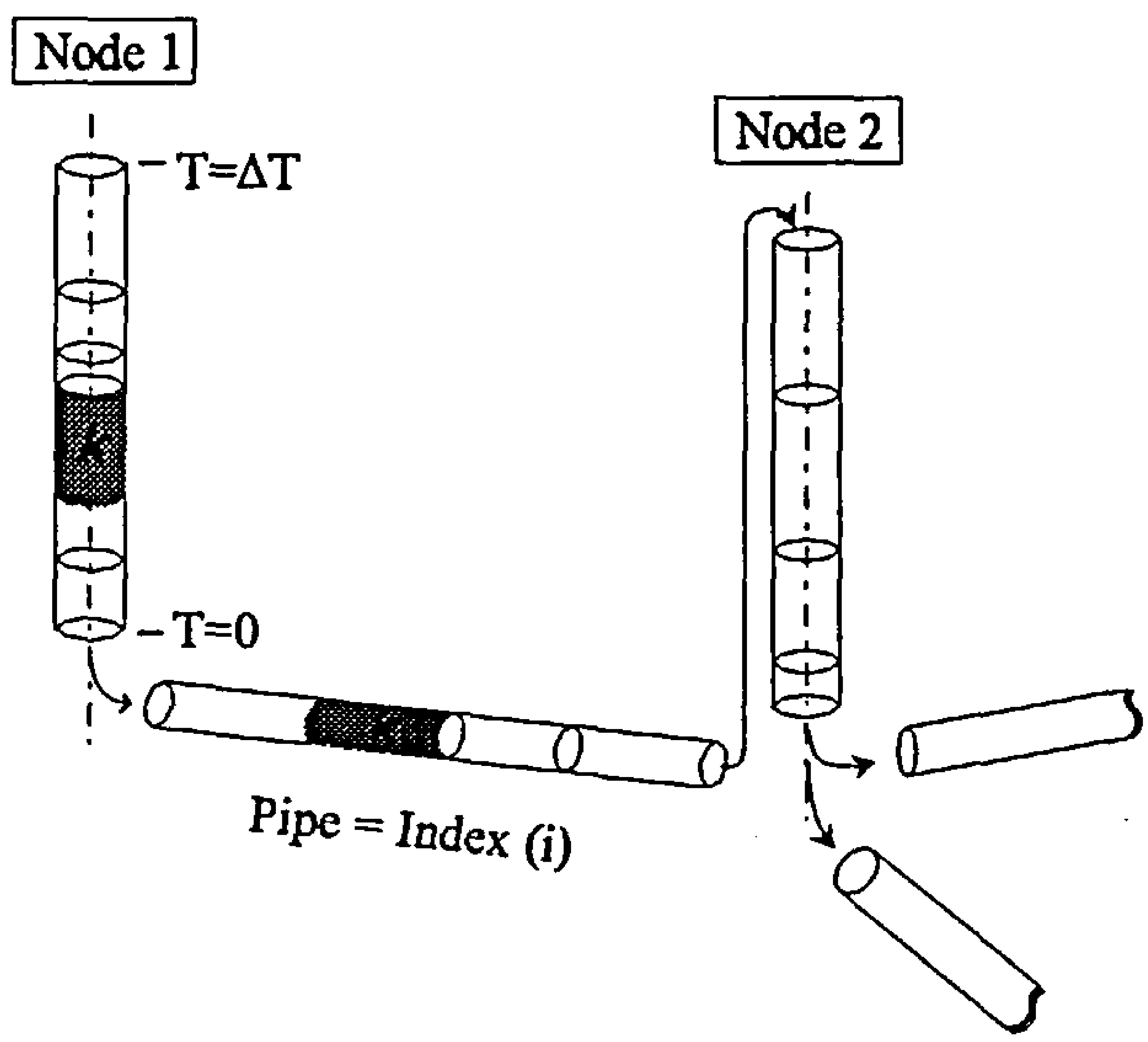


Fig.5.3 - Propagation of flow elements

The scheme can best be described starting from a source node. The method is based on the introduction of sequences or *histories* of concentration values at the source nodes. The variation in concentration at the source is described by a suitably discretised sequence of different values, each with a correspondent duration, independent from, but contained within, the hydraulic time step. These are best visualised as imaginary columns of water at the nodes, as shown in Fig.5.3. The concentrations *injected* at the sources are allowed to change within that time step as many times as required. For each time interval of constant concentration being introduced, a new volume element is injected at the source node into each of the outgoing pipes. At the other end of those pipes, a similar volume is meanwhile being "pushed out" and into an imaginary column at the node. At the end of the hydraulic time step ΔT, a sequence of volumes has been pushed into this column, as shown in Fig.5.3. After all the pipes contributing to this node have been processed, the corresponding number of such columns meanwhile produced is merged, using the respective flow contributions to the node as weights. The node now has a concentration curve ready to be *injected* in all its outgoing pipes and the procedure is repeated.

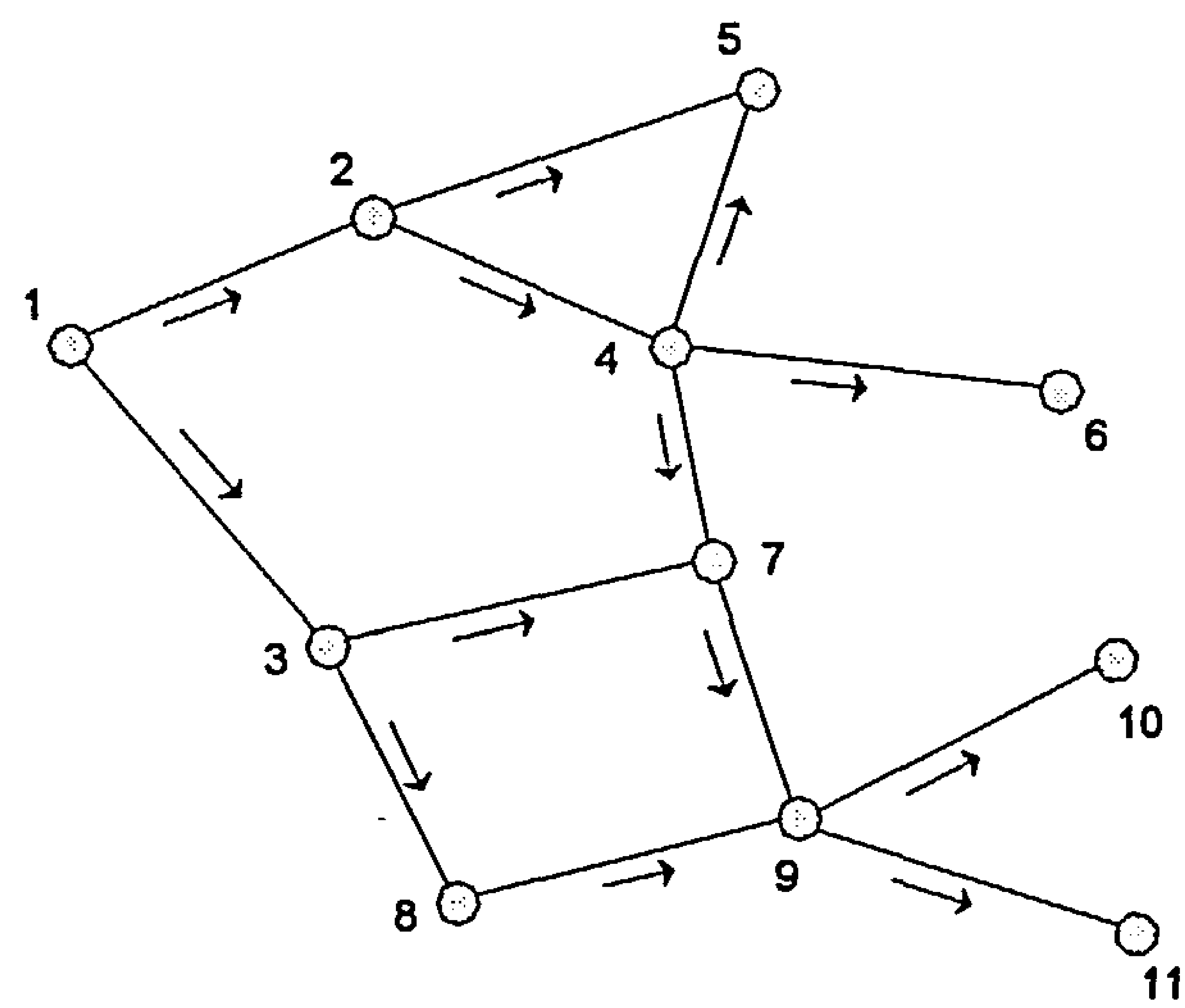


Fig.5.4 - Topological sorting of network nodes

The technique is applied to all the pipes in the network following a sequence of flow precedence, which is defined for each new hydraulic state. For any given hydraulic state, either describing a steady state or a time step ΔT of an extended period simulation, a water distribution network can be thought of as a known directed graph. It is possible therefore to sort all the pipes of the network by topological precedence, in which each pipe must be preceded in the list by all those included in its supply paths. Fig.5.4 shows a simple example of a network which has been topologically sorted for a particular flow distribution scenario.

The numbers next to the nodes represent one of a variety of possible sequences which satisfy flow precedence. It must be stressed that this is a purely topological problem.

The algorithm which will now be formalised is based on that property, and is recursively applied to all the pipes in the sorted sequence defined as above.

Let pipe ij, linking node i to node j, have length L_{ij}. For the given hydraulic time step ΔT, suppose the flowrate is q_{ij} and the velocity is V_{ij}. The variation of constituent concentration along pipe ij is discretised into a number N^{ij} of pipe segments of constant concentration, assuming no longitudinal mixing takes place. The k'th segment has length L_k^{ij} and concentration C_k^{ij}, and the following condition must naturally be met :

$$\sum_{k=1}^{N^{ij}} L_k^{ij} = L_{ij} \tag{5.5}$$

Now let the variation of constituent concentration in the water flowing through node i be described by a discrete time sequence H^i, a history of concentrations values C_l^i and corresponding durations t_l^i, l=1,...,N^i, where N^i is the number of different concentrations through node i during the hydraulic time step ΔT, such that:

$$\sum_{l=1}^{N^i} t_l^i = \Delta T \tag{5.6}$$

For the generic pipe ij as described above, the algorithm begins by adding the history of the upstream node, H^i, to the sequence of segments in the pipe. The new sequence is obtained as follows:

$$C_k^{ij} = C_k^i \,, \quad k = 1,\ldots,N^i \tag{5.7}$$

with corresponding lengths

$$L_k^{ij} = V_{ij} \cdot t_k^i , \quad k = 1,\ldots,N^i \tag{5.8}$$

and

$$C_k^{ij} = C_{k-N^i}^{ij} , \quad k = N^i + 1,\ldots,N^i + N^{ij} \tag{5.9}$$

with lengths

$$L_k^{ij} = L_{k-N^i}^{ij} , \quad k = N^i + 1,\ldots,N^i + N^{ij} \tag{5.10}$$

The number of segments in the pipe is increased by N^i:

$$N^{ij} = N^{ij} + N^i \tag{5.11}$$

and their aggregated total length is now $L_{ij}+V_{ij}\cdot\Delta T$, as a consequence of Eq.(5.5). In other words, there is now at the downstream end of the pipe an imaginary *excess* length of $V_{ij}\cdot\Delta T$, containing a number of pipe segments N_e^{ij} such that:

$$\sum_{k=1}^{N^{ij}-N_e^{ij}} L_k^{ij} \le L_{ij} \tag{5.12}$$

and

$$\sum_{k=1}^{N^{ij}-N_e^{ij}+1} L_k^{ij} > L_{ij} \tag{5.13}$$

are both satisfied.

In case the equality in the first expression does not hold, segment $k = N^{ij} - N_e^{ij} + 1$ is split into two segments of equal concentration, with lengths such that Eq.(5.5) is satisfied for $N^{ij} - N_e^{ij} + 1$. The total number of segments in the pipe is then increased by one:

$$N^{ij} = N^{ij} + 1 \tag{5.14}$$

The N_e^{ij} excess segments will contribute to a new nodal sequence at node j, through which they are effectively flowing during ΔT. Since there may be other contributions to node j, as illustrated in Fig.5.5, this new contributing (nodal) sequence is denoted by $\delta H^{j(i)}$, as it is created in node j by flow originating in node i. Its components $\delta C_k^{j(i)}$ and $\delta t_k^{j(i)}$ are obtained as follows:

$$\delta C_k^{j(i)} = C_{N^{ij}-N_e^{ij}+k}^{ij}, \; k = 1,\ldots,N_e^{ij} \tag{5.15}$$

$$\delta t_k^{j(i)} = \frac{L_{N^{ij}-N_e^{ij}+k}^{ij}}{V_{ij}}, \; k = 1,\ldots,N_e^{ij} \tag{5.16}$$

The number of elements in $\delta H^{j(i)}$ is denoted by $\delta N^{j(i)}$:

$$\delta N^{j(i)} = N_e^{ij} \tag{5.17}$$

Once all the pipes flowing into node j have been processed, the final nodal sequence at j is obtained by applying the principle of complete mixing at the nodes to the contributing nodal sequences $\delta H^{j(i)}$, whose concentrations are weighed by their corresponding flowrates q_{ij}:

$$C^j = \frac{\sum_{i=1}^{U^j} q_{ij} \cdot \delta C^{j(i)}}{\sum_{i=1}^{U^j} q_{ij}} \tag{5.18}$$

where U^j is the set of nodes that directly contribute to node j. Although with the same total duration ΔT, the contributing time sequences $\delta H^{j(i)}$ will generally be expected to differ from one another in the number and duration of its components, as shown in Fig.5.5.

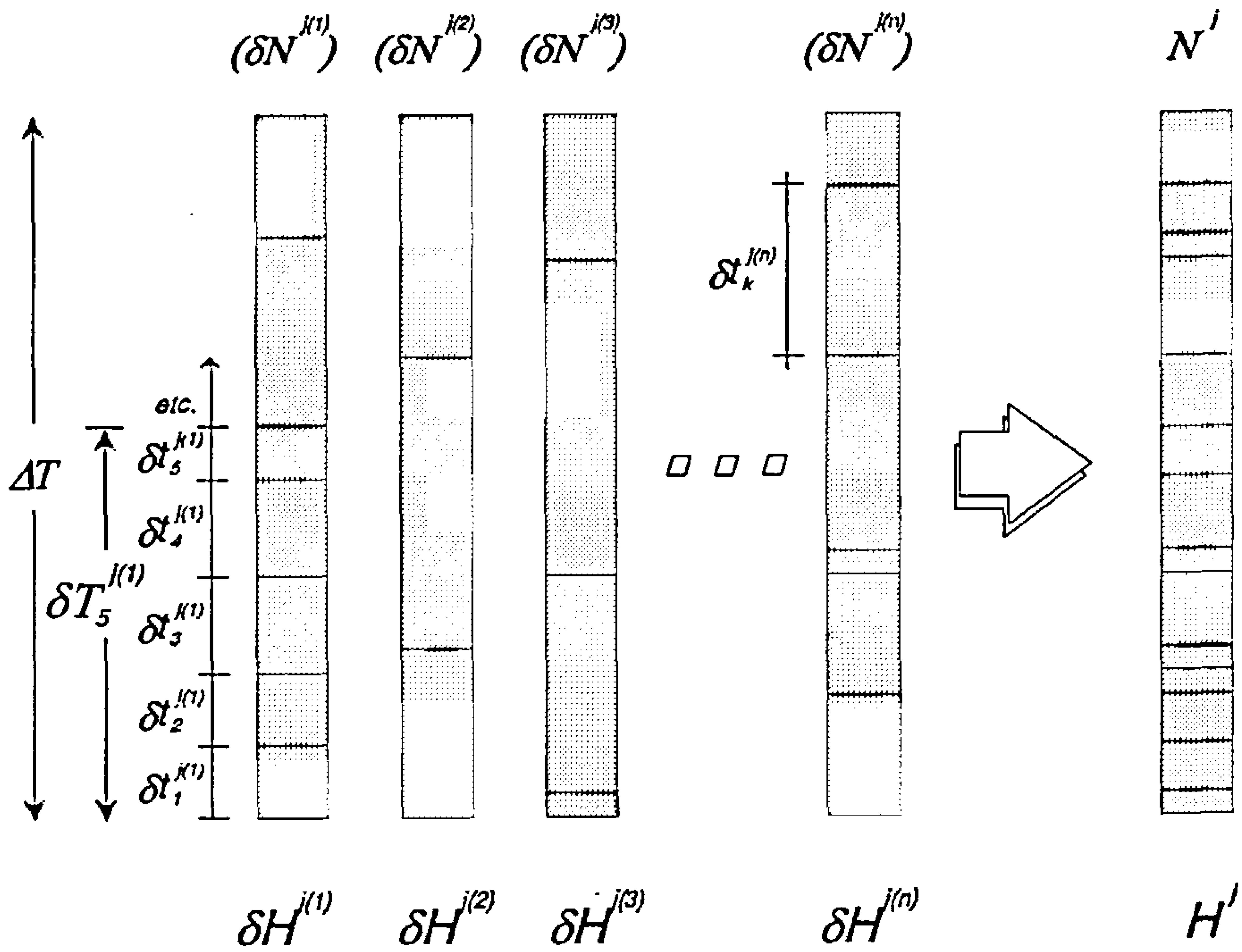


Fig.5.5 - Contributing nodal sequences

In order to carry out the sum $\sum_{i=1}^{U^j} q_{ij} \cdot \delta H^{j(i)}$ it is necessary to reduce all the series to the same time base. The intervals $\delta t_k^{j(i)}$ within the column $\delta H^{j(i)}$ can be changed to a corresponding time co-ordinate through:

$$\delta T_k^{j(i)} = \sum_{m=1}^{k} \delta t_m^{j(i)}, k = 1,\ldots, \delta N^{j(i)} \tag{5.19}$$

The final merged nodal column will have a concentration discontinuity at each point where there is a discontinuity in any of the contributing sequences. The total number of discontinuities in the final column will be N^j. In each contributing column $\delta H^{j(i)}$, a new pair $(\delta C^{j(i)}, \delta T^{j(i)})$ must be created for every change in concentration occurring in any other contributing column at a time T_* that does not coincide with an existing time in $\delta H^{j(i)}$. If T_* is such that:

$$\delta T_k^{j(i)} < T_* < \delta T_{k+1}^{j(i)} \tag{5.20}$$

then a new element is thus created:

$$\delta T_{k+1}^{j(i)} = T_* \tag{5.21}$$

$$\delta C_{k+1}^{j(i)} = \delta C_{k}^{j(i)} \tag{5.22}$$

and all the elements in the series are moved up one position, while the number of elements $\delta N^{j(i)}$ is increased by one. This is repeated until $\delta N^{j(i)} = N^{j}$, so that the following condition is finally satisfied:

$$\delta T_{k}^{j(m)} = \delta T_{l}^{j(n)}, \forall m \neq n \in I^{j}, \forall k \in \delta N^{j(m)}, \forall l \in \delta N^{j(n)} \tag{5.23}$$

The process will eventually cause all the $\delta H^{j(i)}$ series to have the same number of elements, N^{j}, at precisely the same time co-ordinates. The final sequence H^{j} at node j is then given by:

$$N^{j} = \delta N^{j(i)}, \forall i \in U^{j} \tag{5.24}$$

$$C_{k}^{j} = \frac{\sum_{i=1}^{U^{j}} q_{ij} \cdot \delta C_{k}^{j(i)}}{\sum_{i=1}^{U^{j}} q_{ij}}, k = 1, \ldots, N^{j} \tag{5.25}$$

$$t_{1}^{j} = \delta T_{1}^{j(i)}, \forall i \in U^{j} \tag{5.26}$$

$$t_{k}^{j} = \delta T_{k}^{j(i)} - \delta T_{k-1}^{j(i)}, k = 2, \ldots, \delta N^{j(i)}, \forall i \in U^{j} \tag{5.27}$$

This recursive algorithm is applied to all the pipes in the network, following the topologically sorted sequence described before.

The procedure can be summarised in simpler terms:

- the upstream node's sequence is "pushed" into the pipe's upstream end;

- the excess volume "pushed out" at the other end creates a new partial nodal sequence at the downstream node, as it flows through it;

- when all the incoming pipes at the downstream node have been processed in similar fashion, the corresponding partial nodal sequences are merged together, using the incoming flowrates as weights, to yield the nodal sequence at the downstream node.

To trigger the whole process, a set of initial conditions must naturally be defined. For every new hydraulic state (occurring at each new hydraulic time step ΔT) the initial conditions are given by:

- nodal sequences at the sources nodes;

- pipe sequences at all the pipes.

The nodal sequences at the source nodes translate the water quality events whose effect in the network is being modelled. As the S source nodes occupy the first S positions in the topologically sorted list, their nodal sequences will effectively spark off the whole process. Typically, those will consist of water quality parameters which are specified as fixed values for the steady state condition or for each hydraulic time step of an extended period simulation. However, the way in which the model accepts full nodal sequences as initial conditions means that the water quality simulation need not be bound by the size of the hydraulic time step and can effectively calculate much finer changes in the parameters' values.

The pipe concentration sequences are the actual description of the water quality parameter throughout the network. They are either carried over from the previous ΔT or defined as the initial water quality solution to the network. Apart from this latter case, in which the initial discretisation may or may not introduce a degree of approximation (for example in the frequent initial case of no substance in the network, those pipe sequences will consist of only one element of zero concentration, in which case this is an exact initial solution), the method suffers no numerical diffusion whatsoever from one ΔT to the next as the pipe sequences are carried over intact.

This is especially important in pipes where flow direction reversals occur in successive time steps. Since the same pipe segments are used from the end of one time step to the beginning of the next one, the process is modelled as it actually occurs in reality — assuming the sequence depicts the variation of concentrations accurately — without the need for interpolation or other error inducing mechanisms often found in other algorithms.

In other words, the model is driven by the water quality events in themselves, as depicted by changes in concentrations of the substances at stake, as they enter the network, mix at the nodes or react along their paths, and not by a particular (arbitrary) discretisation of the water quality space-time distribution. One immediate consequence is that the model uses the smallest possible number of discrete pipe elements to depict any specific water quality situation.

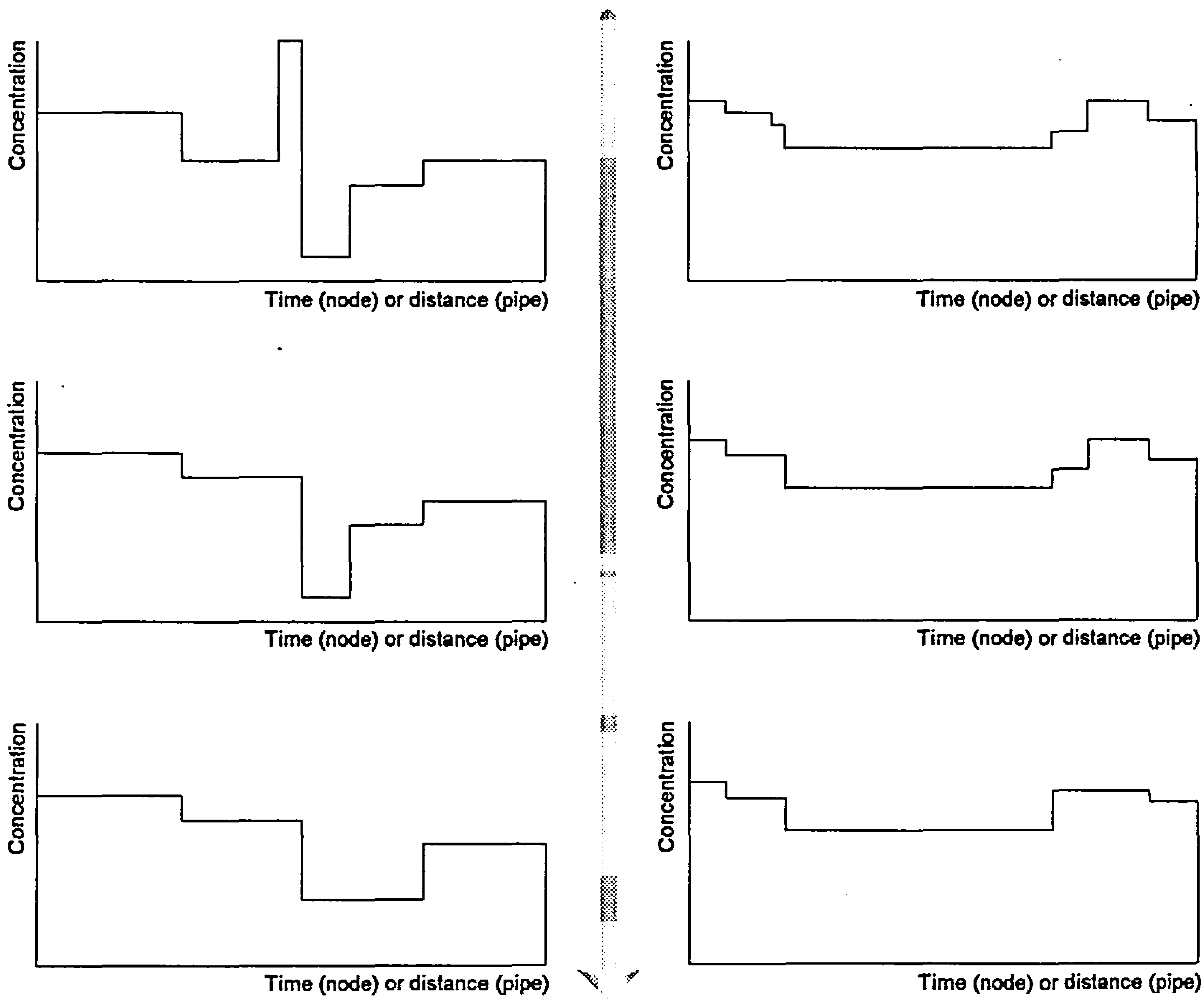
The only potential numerical approximation introduced by this formulation is that which might be caused by the inevitable limitations of the computer implementation. The number of elements in the pipe and node sequences must be kept under given limits which are dictated by the available computing power and memory. However, with new segments created every time the nodal mixing conditions change, there is a possibility that those limits may potentially be reached. What criterion to employ in order to eliminate the excess segments is a somewhat critical area with definite repercussions on the model's accuracy.

One possible mechanism is to merge the shortest element in the sequence — that is, the pipe segment with the shortest length or the nodal segment with the shortest duration — with a neighbouring segment. The neighbour with the nearest concentration is chosen. Aggregation is performed by adding the two lengths or durations and calculating a weighed average of the concentrations. For a nodal sequence, that would be:

$$C_k^i = \frac{t_k^i \cdot C_k^i + t_{k+1}^i \cdot C_{k+1}^i}{t_k^i + t_{k+1}^i} \tag{5.28}$$

$$t_k^i = t_k^i + t_{k+1}^i \tag{5.29}$$

with the number of segments reduced by one and full re-indexing of segments k+1 to N^i. That would be repeated until the limit is satisfied. Fig.5.6 shows the effects of merging using the



a) Short burst

b) Smoothly varying

Fig.5.6 - Shortest segment merging

shortest segment for (a) short concentration bursts and (b) smoothly varying concentrations.

This method is suggested by Liou and Kroon (1987) for a similar problem in their model, and yields perfectly reasonable results when modelling smooth variations in inflow concentrations. However, the method may be less adequate when simulating the effects of short duration events, say for example the accidental introduction of high concentrations of a certain pollutant for a short time. The search for the shortest segment will eventually cause the concentration burst to be rapidly diffused along the direction of flow, an effect which may

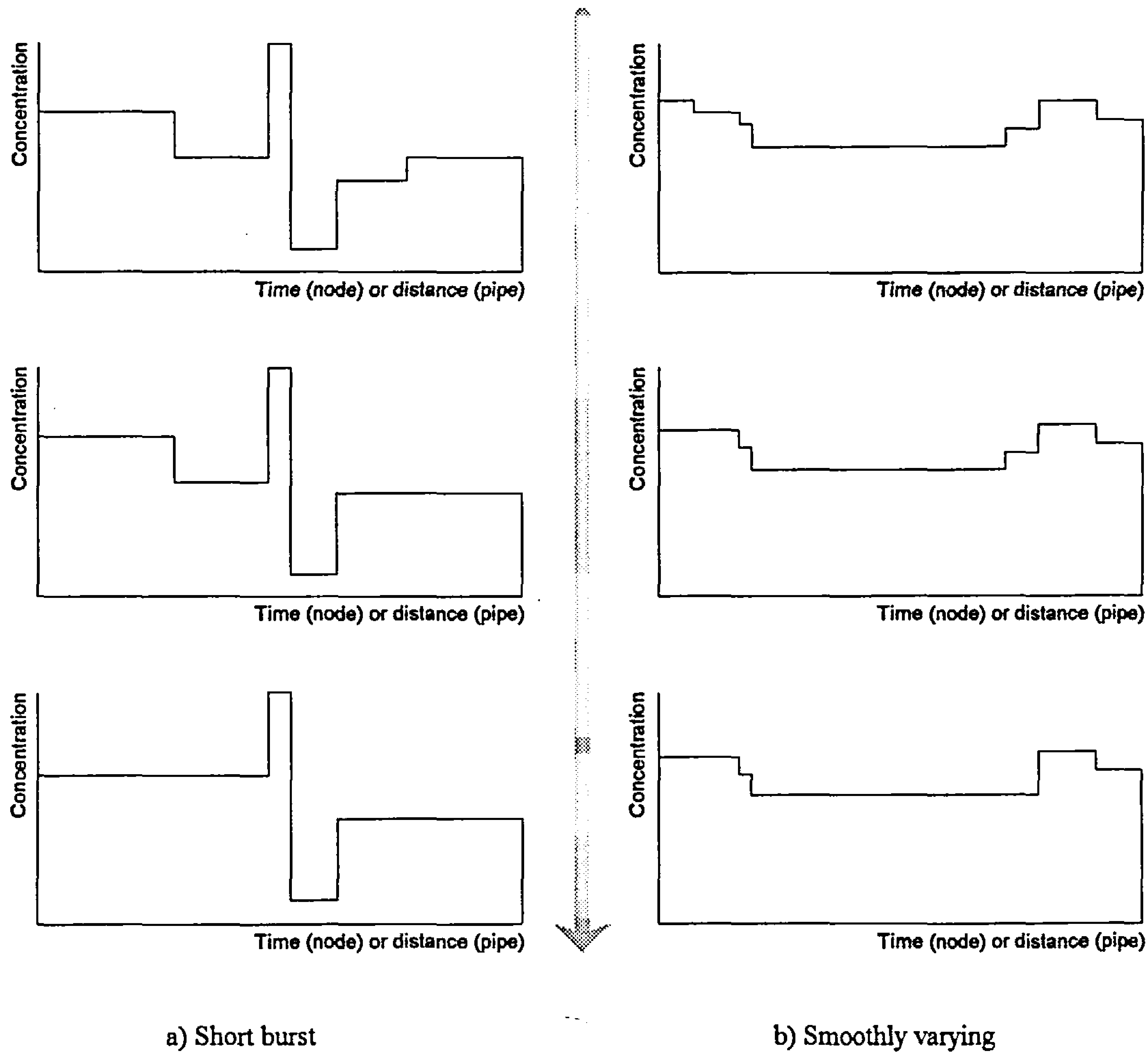


Fig.5.7 - Smallest concentration difference merging

introduce inaccuracies since longitudinal mixing is not intentionally modelled here.

A simple alternative to the shortest segment method is to merge instead the two segments with the smallest difference in concentration. This technique is better suited to sequences with sharp changes in concentration, as shown in Fig.5.7, again for (a) short concentration bursts and (b) smoothly varying concentrations. Although difficult to demonstrate in a short time section such as shown in the figure (in fact, the end result for the (b) case is not too far off the previous figure) the method's natural tendency to smooth out and eventually hide slowly

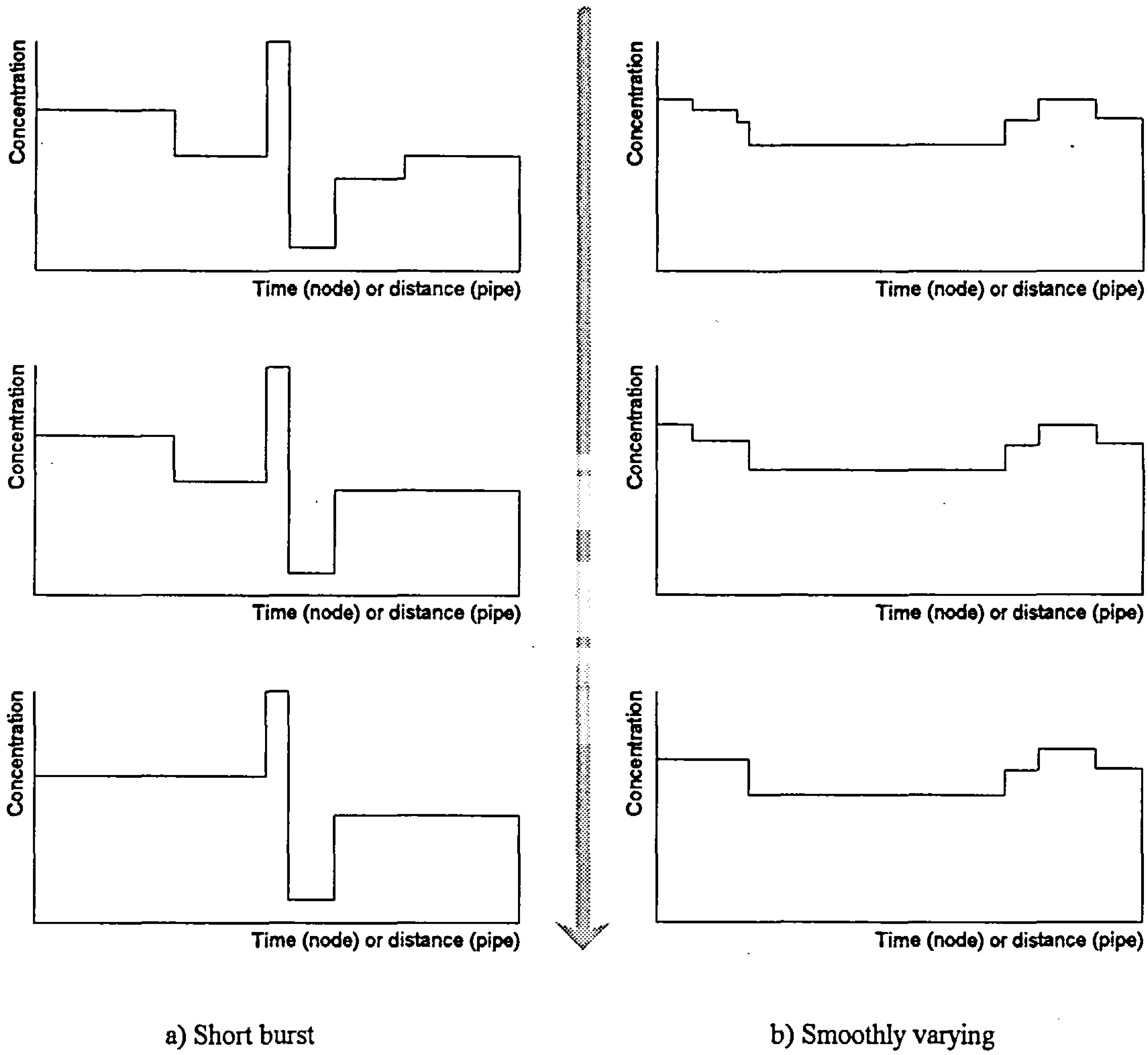


Fig.5.8 - Minimising merging error

varying concentrations may introduce some error if the segments involved have large volumes.

In essence, both solutions try to minimise the area of the concentration's time profile that is modified when merging two segments, by choosing the shortest element respectively along the horizontal and vertical axis. If no considerations are made about preserving certain features of the profile (such as the above mentioned short bursts), then the most accurate method consists in choosing the two neighbouring segments whose combination modifies the smallest area (volume or mass) of graph.

This should represent a good compromise between the two extremes, as Fig. 5.8 shows for the same two cases as before. It does, however, incur in some additional computational effort, which will be wasted in most circumstances given the low magnitude of the errors involved. Further sections will refer to the application of these techniques and discuss their performance.

5.3.3. Transformation models

The model so far described calculates only two of the three processes that make up for the behaviour of waterborne substances in distribution networks, as discussed in 5.2.3.. After the advection and mixing processes analysed, the component still missing is, of course, the transformation process. As mentioned before, that affects only those substances which are not conservative.

It has been seen previously that an equation of the type of Eq.5.2 allows for a suitable model for the transformation process endured by the constituent at stake, as it travels along the pipelines and throughout the network, reacting within itself and with its surroundings, and experiencing decay, growth or other changes, as well as potential combination with other elements.

One of the most important decisions to be made concerning the model is, therefore, what reaction rate function RF(C) to use in Eq.5.2. A variety of models is available, given the diversity of waterborne substances that may be the subject of the modelling effort. Conversely, it is also by manipulating the RF function with suitable transformations that the methodology is able to calculate as well not only the travel time of water to any point in the network but also the contributions of the different sources (if more than one) to any node, as will be seen in the following section.

The classic transformation function, chosen for most of the models proposed in the literature (ex: Burgess *et al.*, 1993, Clark, 1993, Grayman *et al.*, 1988, Liou and Kroon, 1987), is a simple first order transformation process of the type:

$$RF = {}^{dC}\!/_{dt} = kC \tag{5.30}$$

which is solved thus, integrating over $\Delta t = t - t_0$:

$$C = C_0 e^{k(t-t_0)} \tag{5.31}$$

where C_0 is the initial value of concentration at time t_0 and k is the first rate coefficient. The value of k will be negative for modelling decay and positive when representing growth of the substance.

Rossman (1993), proposes a more sophisticated version of this model, accounting for first order kinetics in reactions both within the bulk flow and between the flow and the pipe wall, by means of the following expression for k:

$$k = k_b + \frac{k_w k_f}{R_H\left(k_w + k_f\right)} \tag{5.32}$$

where k_b is the first order bulk reaction rate constant (s^{-1}), k_f the mass transfer coefficient between bulk flow and the pipe wall ($m\ s^{-1}$), and k_w a reaction rate constant ($m\ s^{-1}$) for the pipe wall. R_H is the hydraulic radius of the pipe cross-section, equal to the area divided by the perimeter (half of its geometric radius in the case of a circular cross section).

This type of reaction rate function is often applied to the modelling of residual chlorine. This is a product of disinfection by chlorination at certain points in the network, and is intentionally kept in the water for further protection. As it is typically the main protection agent against microbiological contamination, its decay process is a major concern of network managers who endeavour to keep its levels within the prescribed limits.

In the model described here, that expression is integrated for each pipe element, over the length travelled by it during the hydraulic time step ΔT. In practical terms, that means that equations 5.7 and 5.9 are replaced respectively by 5.33 and 5.34 as follows:

$$C_k^{ij} = RF(C_k^i), \quad k = 1,\dots,N^i \tag{5.33}$$

and

$$C_k^{ij} = RF(C_{k-N^i}^{ij}), \quad k = N^i + 1,\dots,N^i + N^{ij} \tag{5.34}$$

The formulation above shows how an RF function is taken into consideration by the model developed. Two plausible functions are given, equations 5.30-5.31 and the extra sophistication of 5.32, both available in the computer implementation of the model, PERFORMANCE-Q, described further on. Those two functions cover the types most frequently quoted in the specialised literature, with actual applications to the modelling of chlorine residuals described, for example, by Burgess *et al.* (1993), Clark *et al.* (1994), and Tansley and Brammer (1993).

However, while the advection and nodal mixing elements are better known and more easily calibrated, as mentioned in 5.2.6., the transformation function is a field where much work remains to be done in the experimental development and testing of reliable models for the different parameters that are the concern of water quality guidelines. A water quality model is virtually useless for simulating non-conservative substances if that critical component is not valid. One of the most frequent criticisms made to transformation models such as Eqs. 5.30 to 5.32 is that they are single variable (time) and do not take into account the combined effect, on the parameter's concentration, of such factors as flow velocity (which, among other consequences, has a direct bearing on the contact mode with the pipe walls), water temperature and pH, or other substances or elements present in the water.

Accounting for the effect of velocity is straightforward enough once the relationship is known, since the values are readily available, but the implicit suggestion is that the experimental development of transformation functions should be based on multi-parameter models, that is, methods that can handle the concentrations of several different parameters simultaneously.

That possibility is accounted for by PERFORMANCE-Q, which effectively takes an array of concentration parameters (the C matrices in equations 5.7 to 5.27 effectively gain one more dimension to hold the concentrations of multiple parameters) and calculates their propagation simultaneously. In this way, the concentrations of more than one parameter can be made available for calculation of a multi-variable transformation function, where such functions are available.

5.3.4. Travel time and source contribution

Calculating the travel time from a source or point of final disinfection to any point of a water distribution network, or the *age* of the water as it is often referred to, provides a simple, non-specific measure of the overall quality of the water delivered. It can also yield vital information in the identification of *young* and *old* water zones in a system. The calculation is relatively straightforward using a dynamic water quality model, by using the flow velocity values yielded by the hydraulic simulation. It should be mentioned that this is an area where mathematical models really excel over their physical counterparts, providing much higher accuracy and flexibility in studying dynamic situations than the traditional tracer methods.

The model developed in this work can easily accommodate travel time calculations. To accomplish this, it is enough to replace the variable C in equation 5.2 with a time variable T representing the age of water, provided the reaction term RF is set to a constant giving the rate of increase of time *with time*, which of course is 1.

$$\frac{\partial T_{ij}}{\partial t} = V_{ij}\frac{\partial T_{ij}}{\partial x_{ij}} + 1 \tag{5.35}$$

If the time units used for defining travel time are not the same as those in which the flow velocity is expressed, then that constant can be set as the conversion factor between the two.

The initial conditions to be defined are now the initial age of the water at the sources, which would typically be zero if those points correspond to water quality control stations, but may of course be specified otherwise. The assumption of weighed mixing at junction nodes is kept, which means that the travel time obtained at any point for which there is more than one supply path is the weighed average of the travel times of the different source paths of the water at that point.

An equally simple but often crucial calculation, when dealing with systems with multiple sources and blending of waters from diverse origins, is the percentage of water reaching any particular point in the network from any other node. This analysis is normally directed at source or storage nodes and termed *source contribution,* but in actual fact the procedure described below applies to the fraction of water originating from any source in the network. Further to showing how water from a given source blends with that from other sources, the fact that the model is able to simulate dynamic extended periods allows for the analysis of how the spatial blending patterns change over time under varying demand conditions.

In this case, the variable C in equation 5.2 is replaced by a source contribution value S, relative to the designated origin node[4] being analysed. This quantity is best expressed between zero and one (or in percentage terms) and is always initialised as 1 (or 100%) at the said origin node, while at all other source nodes its value is zero. The reaction term RF is in this case set to zero, since the fraction of water originating from a particular node carried by any segment is clearly "conservative", in that it does not change with the flow along a pipeline.

$$\frac{\partial S_{ij}}{dt} = V_{ij} \frac{\partial S_{ij}}{\partial x_{ij}} \tag{5.36}$$

The tracing of source contributions with this dynamic model effectively extends the capabilities of the previously mentioned microflows model (see preceding chapter) to the extended period simulation domain.

5.3.5. Modelling of other network components

This text has so far been concerned with modelling the movement and transformation of water quality parameters along the pipelines and through junction nodes. Water distribution networks naturally include other components. It seems reasonable to assume that pumps, valves and similar devices may have a negligible effect on the advection, mixing or transformation processes, since flow through them is considered to be instantaneous. There is however a class of devices which will most certainly influence all three of those processes, sometimes quite dramatically: storage facilities. The time of permanence of each volume of water in reservoirs, storage tanks and various types of chambers, frequently known as *residence times,* are often a concern for water distribution managers because of their potential negative effect on the water quality. While long residence times are not always avoidable — after all, the purpose of storage devices *is* to keep the water for a certain amount of time — it

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4 Although the analysis will typically focus on actual source nodes, any node in the network can be taken as an origin node and its influence on any other node can be studied through the procedure.

is imperative that those are known as best as possible and their effect on the quality of the water analysed and if possible modelled. Indeed, the modelling of storage facilities for water quality purposes is nowadays one of the most delicate aspects of water distribution networks analysis.

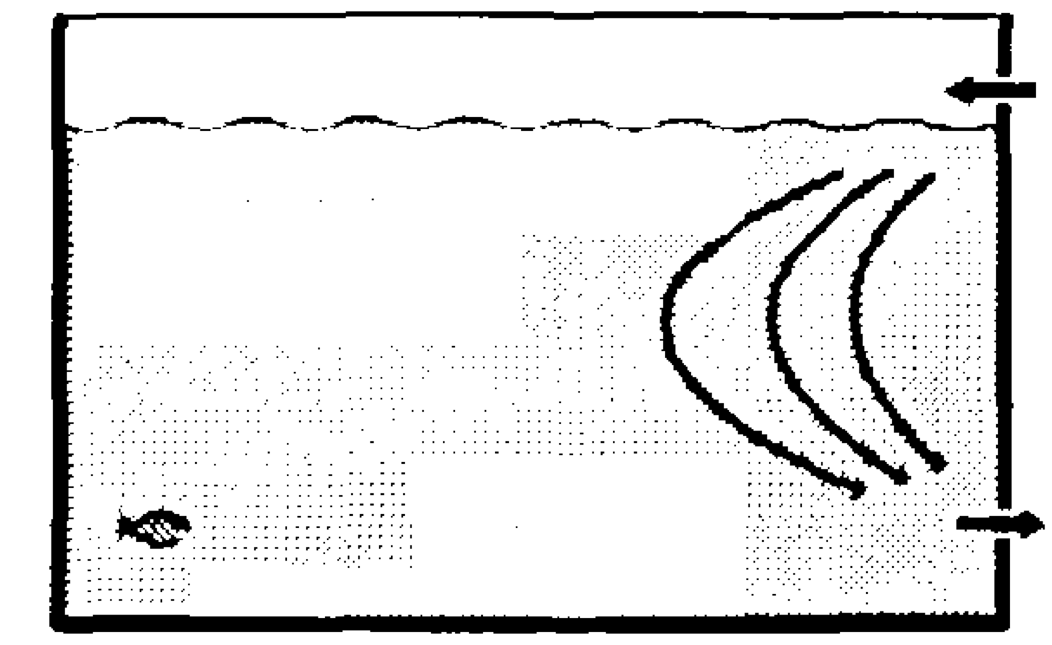
A simplistic approach, which is nevertheless found in most of the published methodologies (e.g. Males *et al.*, 1985, Clark *et al.*, 1988, Liou and Kroon, 1987, Cohen, 1990, Clark *et al.*, 1993, Ulanicki, 1993), consists of assuming the storage device as fully mixed at all times. The concentration of incoming flows are therefore diluted against the existing one by calculating the weighed average of the two masses over the time period considered, and the outflow carries the same fully-mixed concentration. The following equation can be written for a variable level reservoir:

$$\frac{d(C_r VOL_r)}{dt} = \sum Q_{ir} C_{ir} + RF_r(C_r) - \sum Q_{r0} C_r \tag{5.37}$$

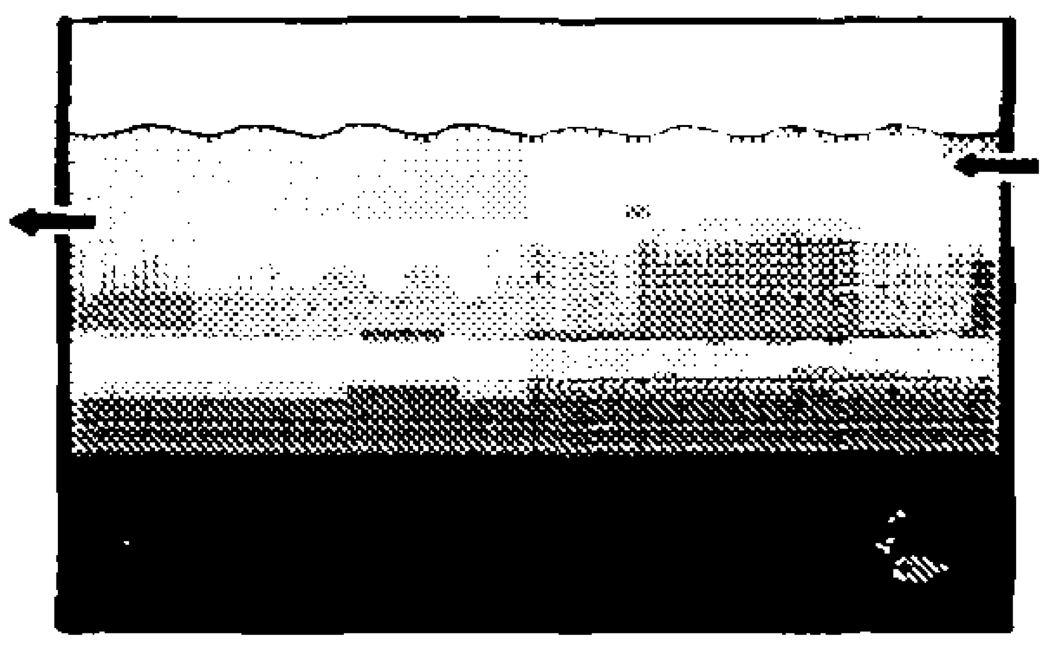
where VOL_r is the volume of water stored, C_r the fully-mixed concentration, Q_{ir} and C_{ir} the incoming flows and their concentrations, and Q_{ro} the outgoing flows.

Even though it is a simple, easily-modelled procedure, and as such is the one included in PERFORMANCE-Q, the fully mixed model can be criticised for not taking into account certain phenomena which are very common in reservoirs, such as stratification, plug flow, short circuiting and other patterns of slow flow from the inlet to the outlet (Fig.5.9).

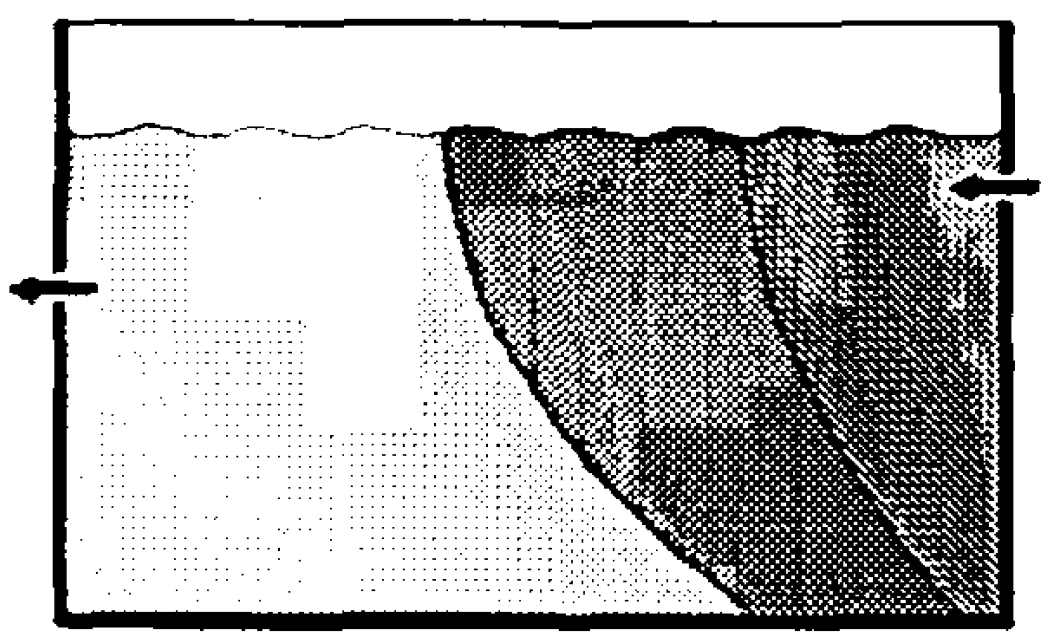
The more important the effect of particular reservoirs on the water quality of the network, the more necessary it will be to discard a simplified solution and develop dedicated reservoir simulation models. An example of such a solution is given by Clark *et al.* (1991), with a multi-compartment model yielding good results for the Cheshire service area of the South Central Connecticut Regional Water Authority (SCCRWA) in North America.



Short-circuiting



Stratification



Plug flow

Fig.5.9 - Reservoir flow

The multi-compartment model is a sophistication of the fully-mixed reservoir model, dividing the total volume into a number of fully-mixed compartments. In this case, two compartments are used as shown in Fig.5.10. The smaller, primary compartment is a fixed-volume, fully-mixed reactor as described previously, and covers both the inlet and the outlet to the reservoir, which means all water enters and leaves through it. The larger, secondary compartment is also fully-mixed but has variable volume.

The procedure is controlled by a single variable, the size of the primary compartment. The calibration of such a model must be done carefully and the primary compartment sized in the face of field sampling results. As shown in Fig.5.10 for two different inlet/outlet arrangements, it provides a simple way of modelling short-circuiting and stratification. The model can be used with more primary compartments, although the extra sophistication will also imply a heavier sampling programme to allow for reasonable

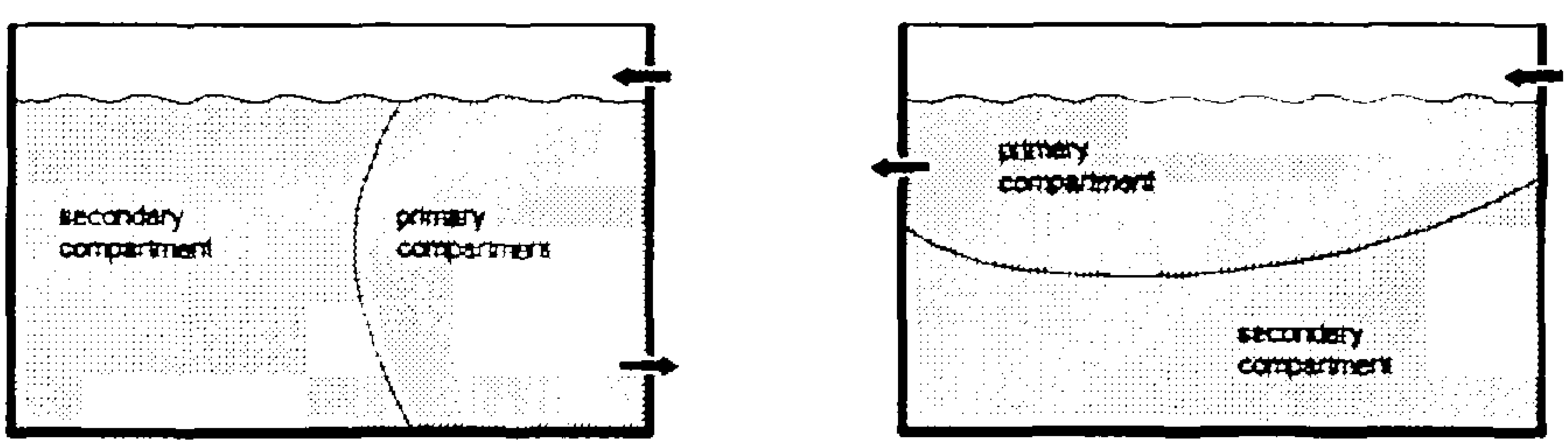


Fig.5.10 - The multi-compartment model for two different inlet/outlet configurations

validation.

It would not require a great deal of imagination to adapt this type of model to a plug-flow situation. It would suffice to model the flow as entering and leaving each compartment in succession, with the last compartment discharging directly to the reservoir outlet. If taken to the extreme, this would lead to a finite-element type solution.

In actual fact, to be consistent with what was argued before about modelling advection along the pipelines, the reservoir plug-flow situation would best be modelled by considering it to be a pipe, albeit with a (likely) variable cross-section along the direction of travel, and applying exactly the same methodology as for the rest of the network. It could be argued, however, that the resulting low velocities and large contact fronts between each segment would invalidate the basic assumption of no longitudinal diffusion.

To summarise, there is no shortage of possible reservoir models to try and cope with the diversity of situation that may arise. However, each particular case must be taken in isolation, developed and calibrated individually. For a-priori inclusion in a general purpose model such as the one described here, it is thought preferable to use a simple fully-mixed tank model.

5.3.6. Further remarks on water quality modelling

Having presented the subject of water quality modelling and having described in some detail the model developed in the present work, there are a few important points which should at this point be emphasised, regarding the general use of such tools.

The first concerns the level of discretisation of the network model that supports the water quality model. For most hydraulic purposes it is reasonably possible to simplify the network, by considering only pipes above a certain size or disregarding certain links, say with very slow velocities or negligible flow. Such simplifications, when properly applied, help reduce the human and computational effort without decisively hindering the validity of the model.

For water quality modelling, however, any link assumes an extra significance which must be taken into account. Very slow velocities are now rather important, and any flow, however small, may be the vehicle of a contamination accident, for example. The very propagation model will lose significance if any link is disregarded.

The second aspect regards the danger inherent to the high degree of sophistication of a water quality model. The calibration of the original hydraulic model is already a rather complicated affair, with never any guarantee of success. Such models may be increasingly easy to set up and run with the modern water simulation packages — optimistic claims of models developed "...within minutes..." are all too frequent in advertising — but any technical manager with hands-on experience of modelling real networks will testify to the arduous, long process of reaching a stable and useful hydraulic model, and maintaining it that way for any reasonable length of time.

Water quality models add further degrees of complication to that uncertainty. Calibrating the advection and mixing processes is less than straightforward even for a fully-calibrated hydraulic model. To that it must then be added the calibration of the transformation process, a definitely difficult field.

The utility of water quality models is unquestionable, but all precautions must be taken to insure that their validity is as good as can be afforded, and that the magnitude of the errors generated is at least reasonably estimated.

5.3.7. Computer program PERFORMANCE-Q

The procedure described above was implemented in a set of FORTRAN routines which form a stand-alone program called PERFORMANCE-Q. The routines were organised in the same format as the global performance evaluation system to enable direct linking to it. The flowcharts in Fig. 5.11 and 5.12 contain a simplified description of the program's structure.

The program takes as input data a flat file of hydraulic results, from a previously run hydraulic simulator, containing:

- The network description, including pipe sizes and lengths and reservoir volume curves;
- the hydraulic solution for the period concerned, including flows and velocities in pipes and the length and number of time steps;

A second input file will contain the sequences of initial values for the source nodes, either as concentrations of one or more substances or as initial travel times. The program also takes a number of user-definable options, including:

- type of simulation — for concentrations, travel times or source contributions;
- in the case of concentration calculations, the choice of transformation model and its parameters, such as decay or growth rates;
- limits for the number of segments in pipes and nodes;
- type of aggregation scheme;

The results produced by the program are described in tables of nodal concentrations, travel-times or source contributions, for all the time steps included in the original extended period simulation. For concentration simulations, a mass-balance calculation is carried out that estimates the difference between the total constituent masses entering and leaving the network over the period concerned. The main use of this latter feature is to check the model's accuracy, for which it must be applied to a conservative substance and over a period of time greater than the longest travel-time in the network.

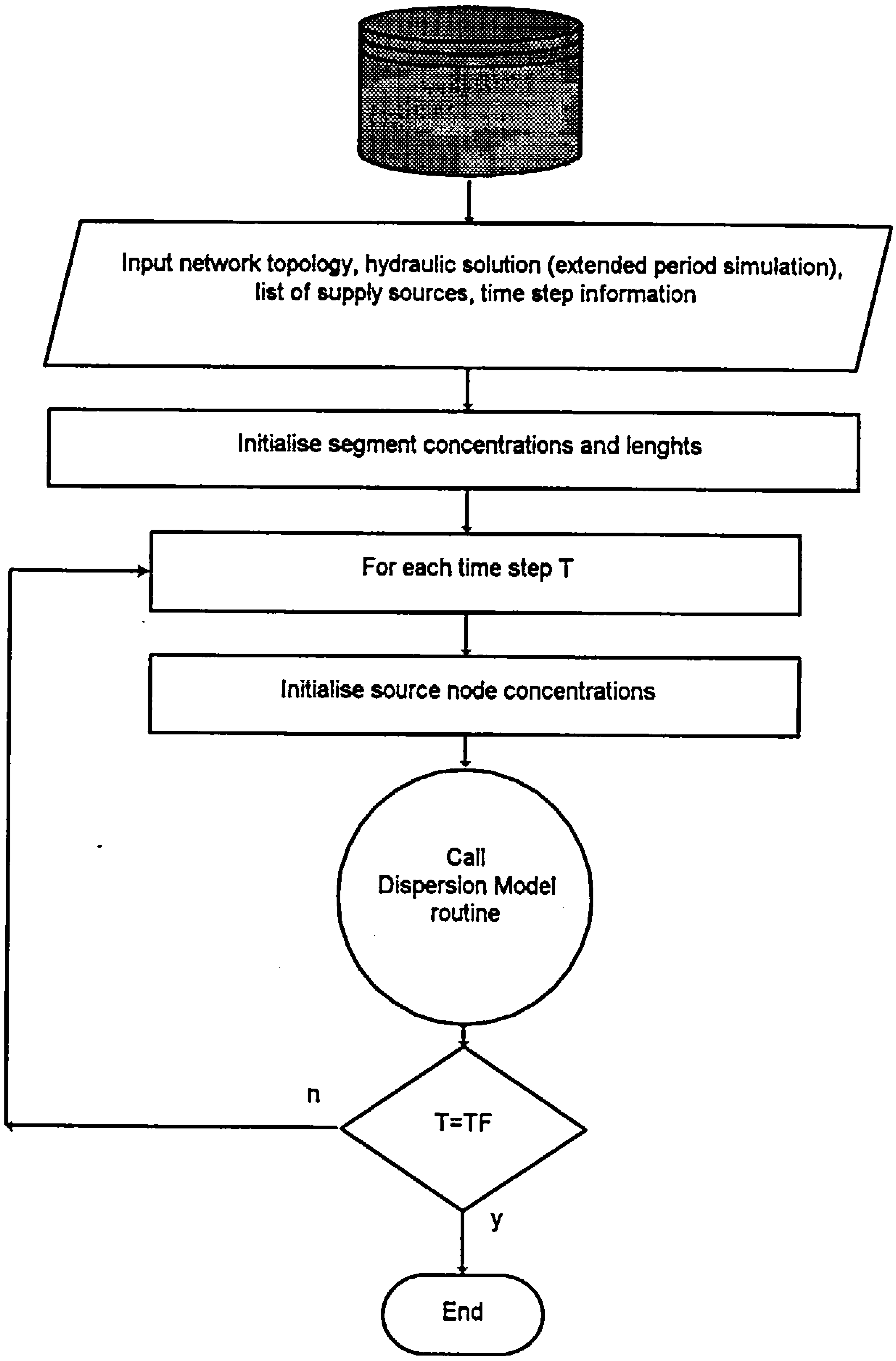


Fig.5.11 - Flowchart of Performance-Q

Dispersion Model routine

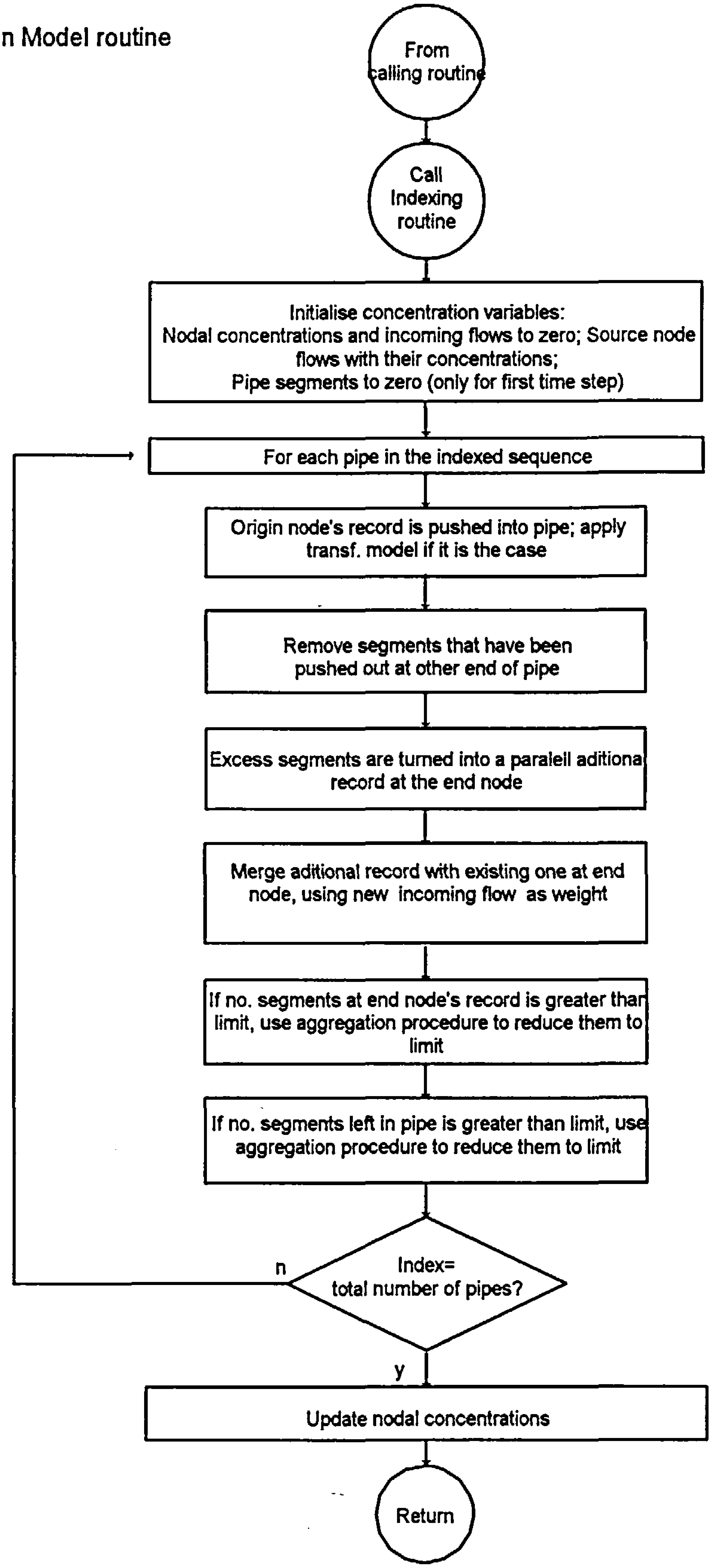


Fig.5.12 - Flowchart of Performance-Q (contd.)

5.3.8. Application examples

A few examples will now be shown to illustrate the use of PERFORMANCE-Q. A first case study will use imaginary test networks to demonstrate the numerical suitability of the method and the different segment-merging techniques. The second case shows the East Edinburgh distribution network and calculates chlorine residual profiles and travel times. The third case compares an established water quality model from the literature with the results from PERFORMANCE-Q.

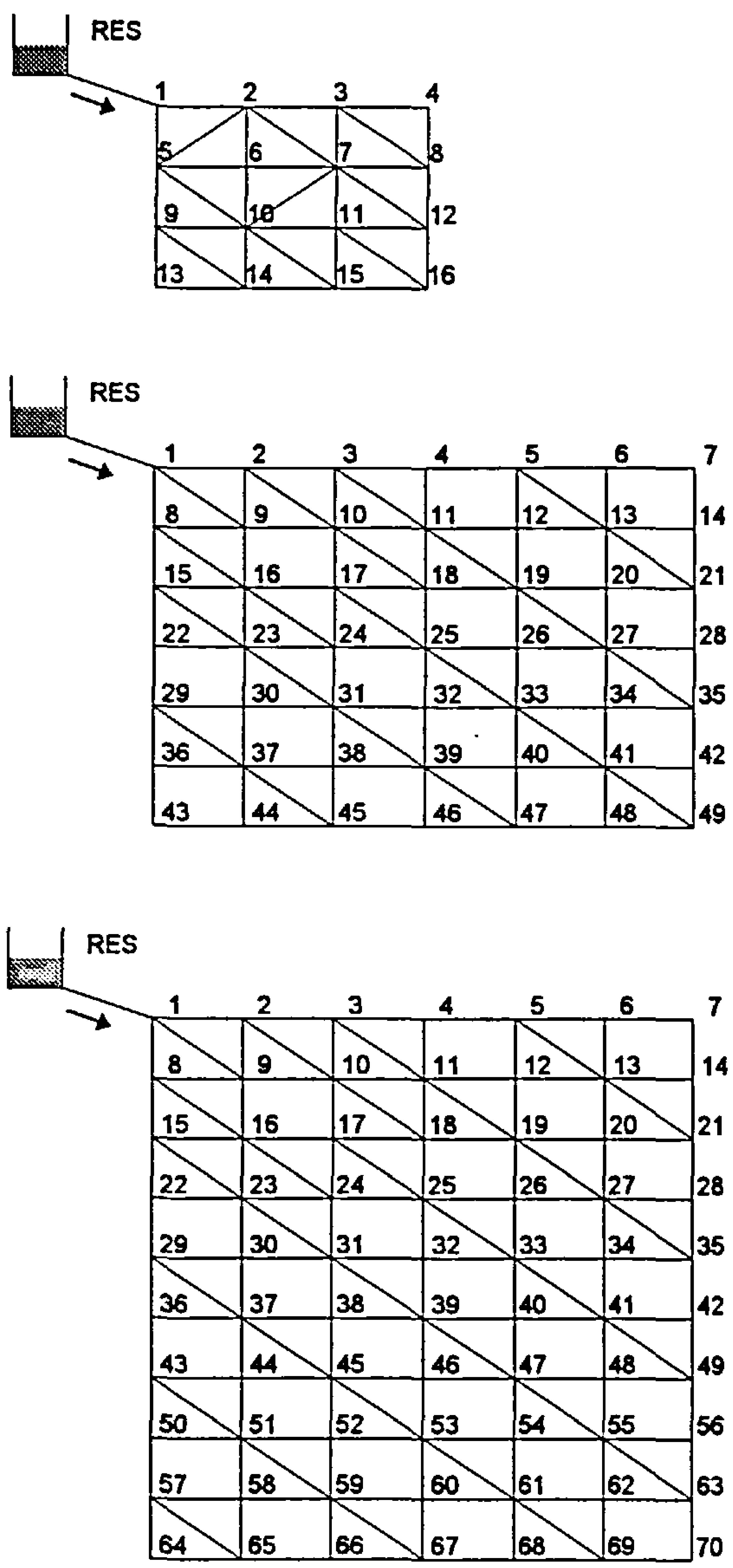
Case 1 - Test networks

Fig. 5.13 displays three test networks with varying number of links but similar pipe sizes. The networks' topological data are included in Appendix A. One of the uses of this type of theoretical network is to assess the variability of numerical errors. A simple verification consists in carrying out a mass-balance calculation for a steady-state hydraulic solution, with an initial injection of a conservative substance, run for a period of time long enough for all the material to flow out of the network, and comparing mass in with mass out. That length of time is determined by calculating travel times for the same hydraulic solution and then running the main simulation until a period longer than the longest travel time has elapsed after the end of the source "injection".

That mass-balance calculation is one of the default results of PERFORMANCE-Q, which keeps track of the mass of substance entering the network per unit time (source concentrations times the corresponding flows) and leaving it (nodal concentrations multiplied by nodal demands). The final result included in PERFORMANCE-Q's output is the relative mass gain, or error, calculated thus:

$$\bar{\varepsilon} = \left| \frac{\sum M_{OUT} - \sum M_{IN}}{\sum M_{IN}} \right| \tag{5.38}$$

where $\overline{\varepsilon}$ is the relative error, M_{OUT} is the mass leaving the network and M_{IN} is the mass entering the network.



Test network 1

16 nodes
34 links

Test network 2

49 nodes
106 links

Test network 3

70 nodes
154 links

Fig.5.13 - Test networks

The graph in Fig.5.14 shows the variation of numerical error with network size, for the three test networks. In each one a 48-hour steady-state simulation was performed, with a 24-hour long injection of the same (conservative) concentration at the start. The flows are proportional to the number of nodes since the nodal demand is a constant. The total mass injected is 18.72 kg, 58.36 kg and 84.79 kg for the 3 networks. The mass-balance error is respectively 0.0087E-3, 0.045E-3 and 0.0981E-3. Although increasing with the number of links, it appears to do so in a roughly direct proportion, and in any case is kept at acceptable values.

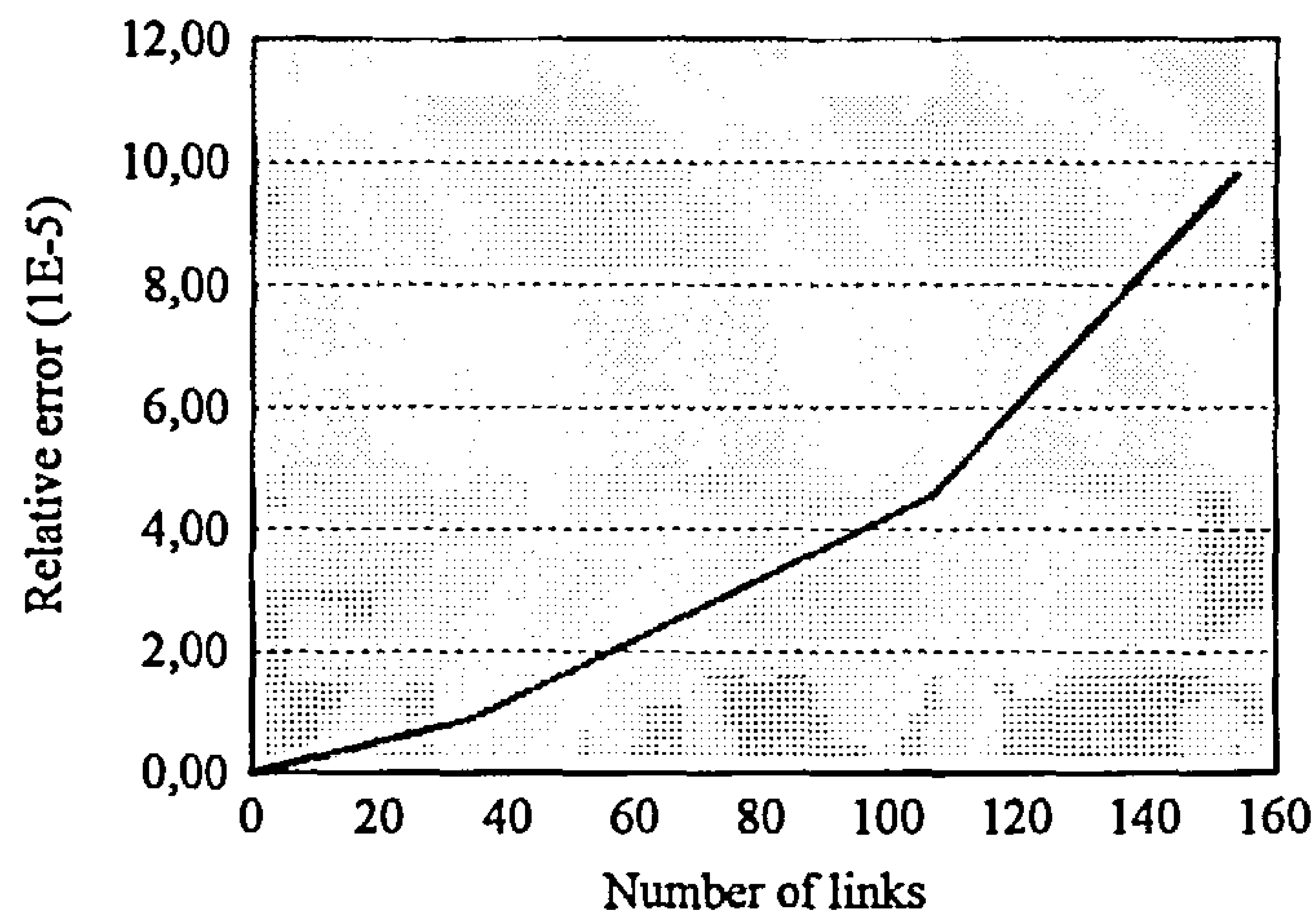


Fig.5.14 - Variation of numerical error with network size

Figure 5.15 shows a comparison between segment merging methods – shortest segment, smallest difference and minimum area modification – for a short burst load injected at the source of test network 3, again with a steady-state simulation running long enough to allow all the mass to flow out of the system. The graphs shown are the concentration time plots at node 42. The second and third methods were expected to perform better than the first for this short burst case. The exact solution was calculated for comparison purposes.

In real terms there is not much difference between the three approximations in terms of how the concentration burst is depicted, even though the smallest difference method does seem to be closer to the exact solution. This was verified in most short-burst tests conducted, and the larger or more complex the network and longer the test, the more noticeable the effect. Some information can be drawn from the mass-balance results, which gave 0.000891 for the shortest segment and 0.000508 for the smallest difference, thus confirming the above impression.

In terms of mass-balance accuracy, the minimum modified area method is in fact the most precise, giving 0.000419 for the relative error in this particular case and generally the best results in other tests. The penalty in terms of computer run-times is however rather noticeable, and increases markedly with length of test or network size and complexity, to a degree that suggests there may be no effective gain in using the method.

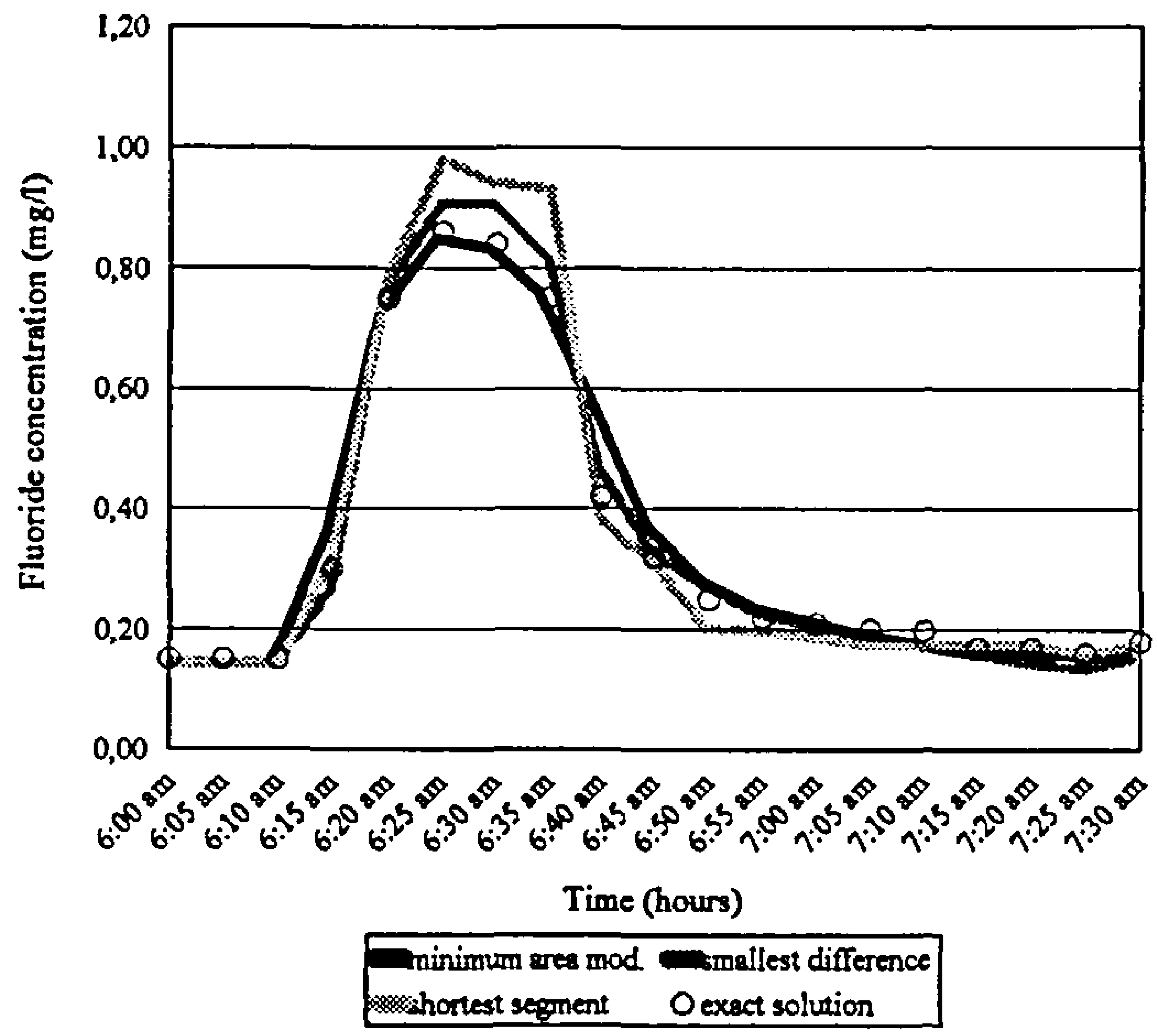


Fig.5.15 - Concentration profiles for different merging methods

The results for smoothly varying concentration tests are more uniform, still marginally rewarding the minimum modified area method in terms of mass balance, but confirming the extra calculation load. In this case the two other methods have yielded invariably similar degrees of accuracy.

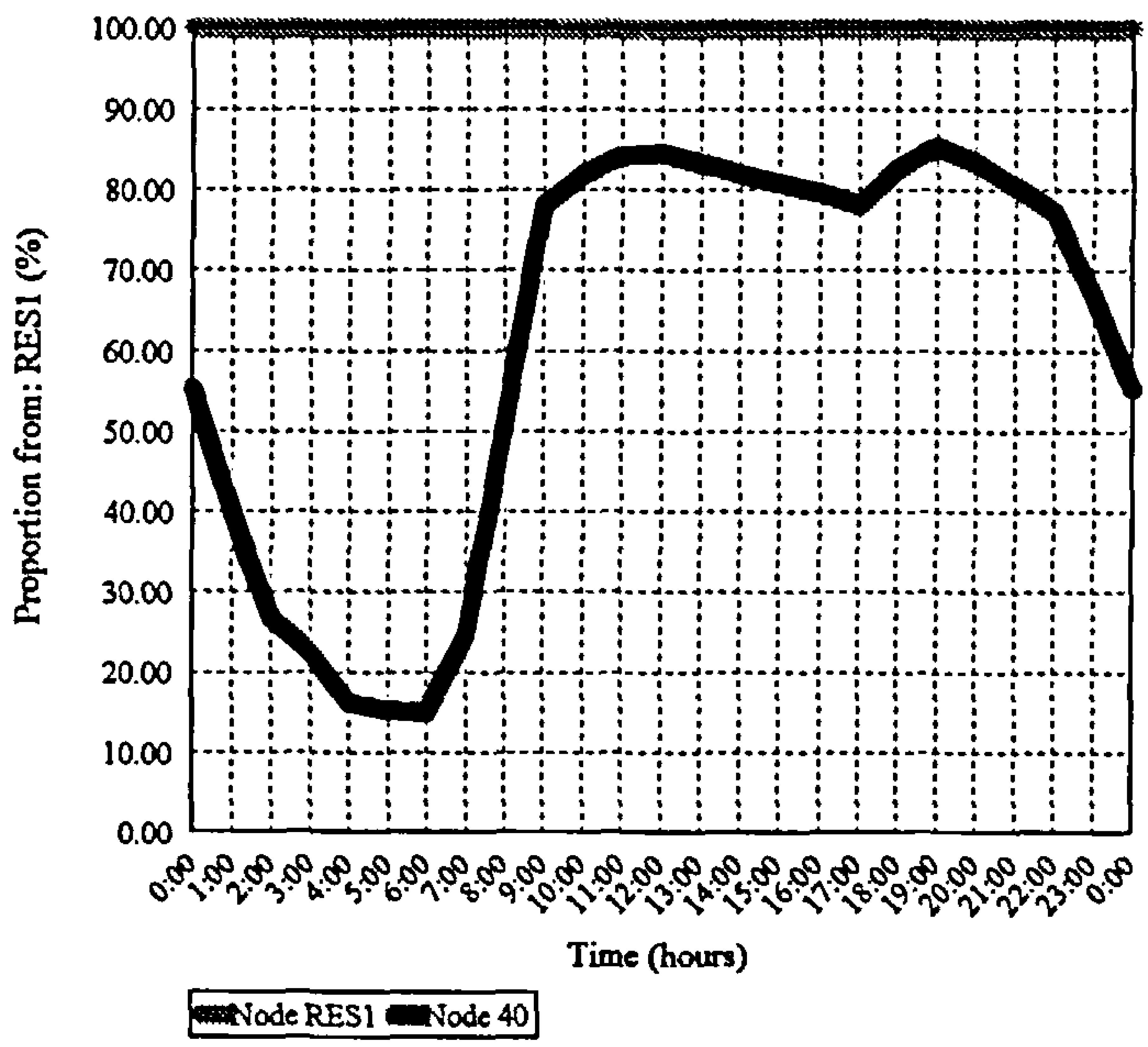


Fig.5.16 - Source contribution profiles (source node and network node)

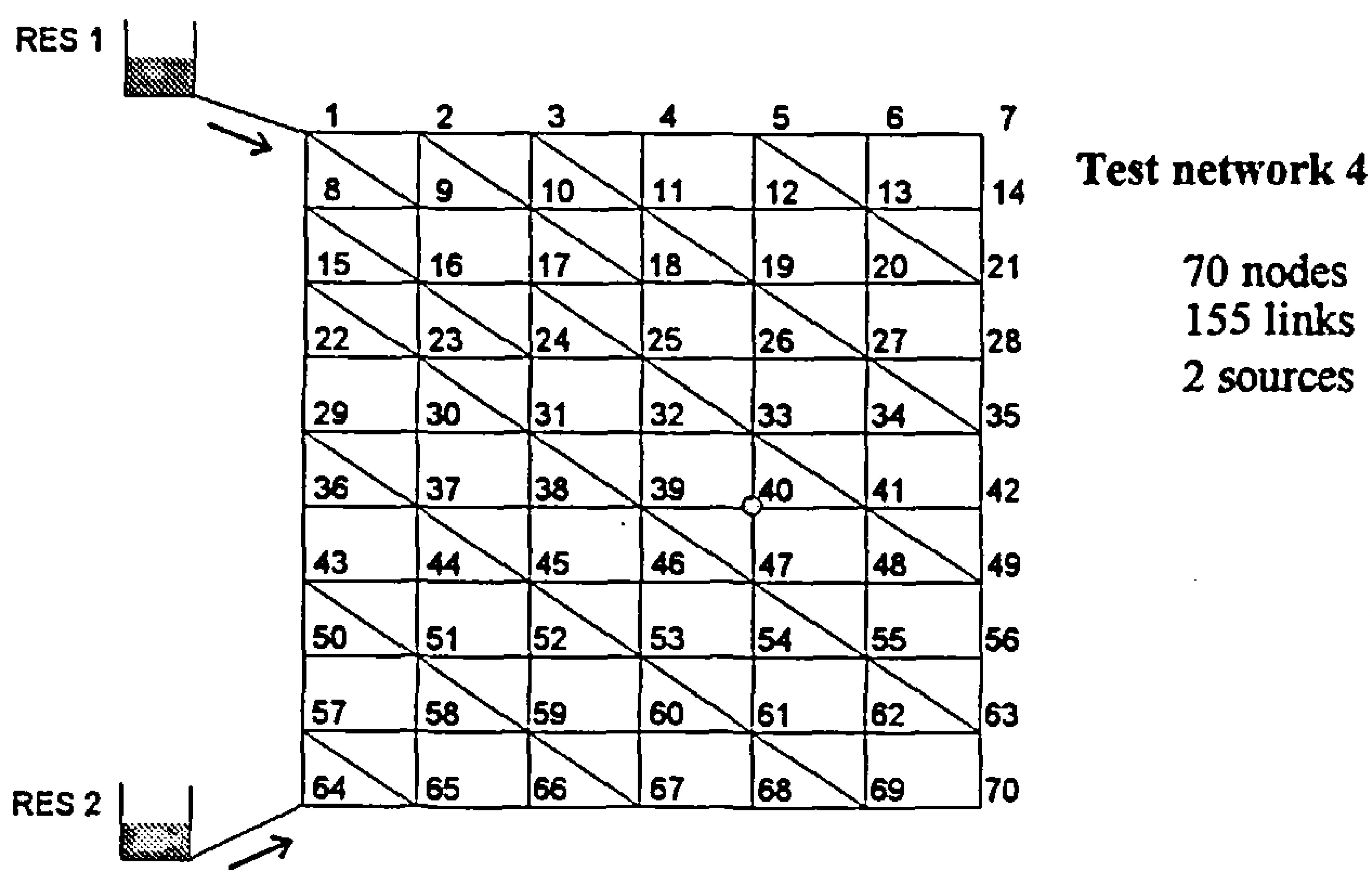


Fig.5.17 - Multi-source test network

Figure 5.16 shows an application of the source contribution facility, with the concentration time plots at a source node (RES1) and at a network node (40) displaying the percentage of water coming from one of the sources (RES1). This test was run with a modified version of test network 3 featuring an extra reservoir as shown in Fig.5.17., and using a typical 24-hour simulation.

Case 2 - The East-Edinburgh distribution network

The distribution network for the eastern part of the city of Edinburgh (Fig.5.18) will now be used to exemplify the simulation of a hypothetical disinfection study.

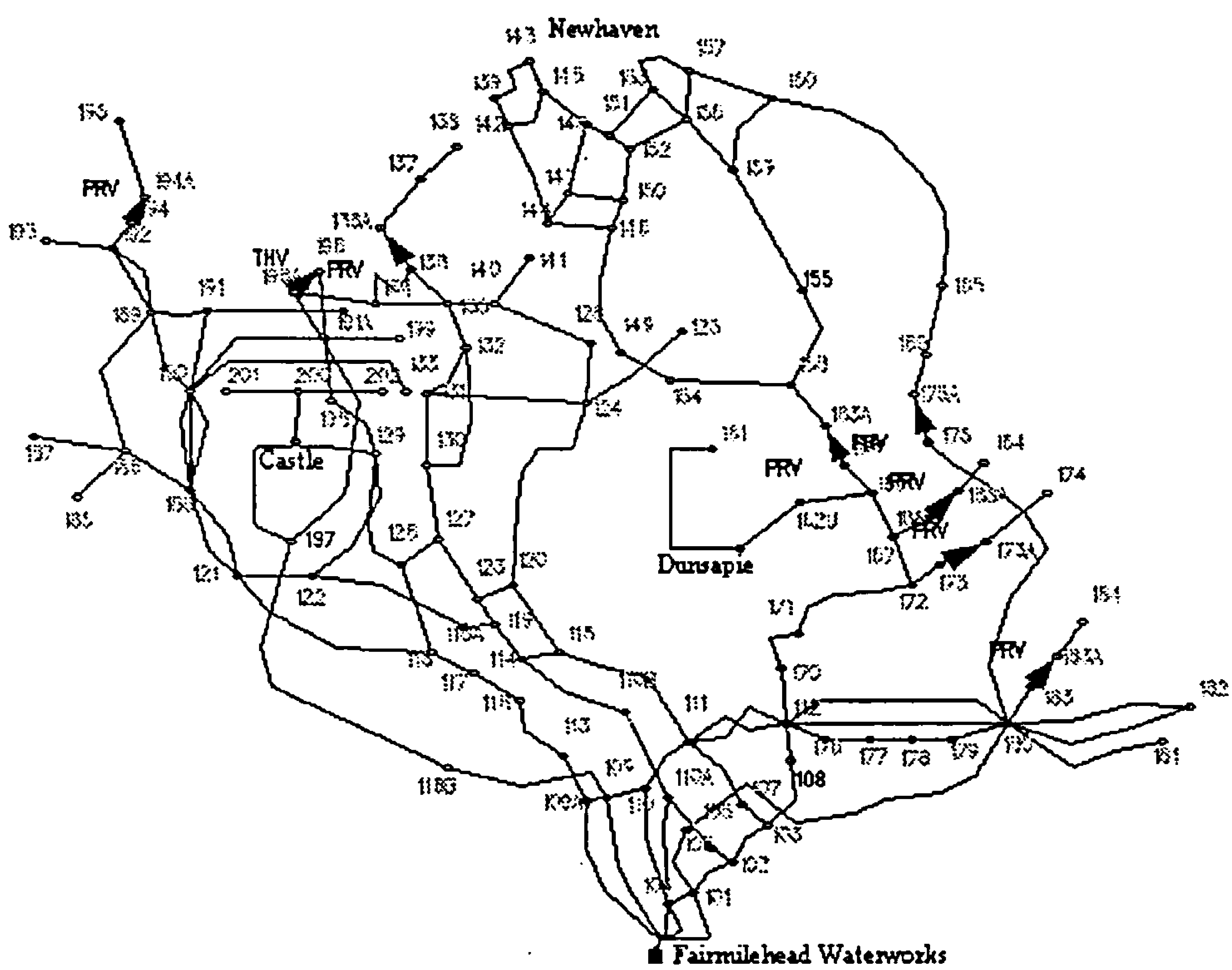


Fig.5.18 - East Edinburgh water distribution network schematic

The main reasons for disinfecting drinking water are to ensure the destruction of pathogens, to create and maintain a protective shield against pathogens entering the distribution system, and to suppress bacterial re-growth in the pipe environment. Because of the importance of disinfection in safeguarding the hygienic quality of potable supplies, it is essential that the concentration of disinfectant should be monitored and followed in great detail. The careful simulation of such concentrations, together with frequent field monitoring, helps improving the placement of disinfection stations and optimising the dosage rates.

Chlorine is the most frequently used disinfectant in drinking water treatment. In addition to disinfection efficiency, it has the ability to provide a persistent residual for continued microbial control after the water leaves the chlorination chamber. The free chlorine residual is a non-conservative substance which decays over time and along its network path. It is recommended (WHO, 1993b) that the concentration of chlorine residuals be kept between 0.20 to 0.50 mg/l throughout the entire water distribution system.

This example shows a simulation of chlorine residual propagation throughout the East Edinburgh distribution system[5], whose data are given in Appendix A. The hydraulic solution used corresponds to an average, weekday demand profile (the maximum load factor for the aggregate demand is about 1.5 of the average daily value), with a typical operational configuration running by gravity from the Fairmilehead reservoir, with the Castle tank by-passed and Dunsapie reservoir in use.

The water introduced at the source (Fairmilehead waterworks) is simulated as containing a constant chlorine residual concentration of 0.5 mg/l, inside the prescribed limits. A simple first-order transformation model (Eq. 5.31) is then used with an average k value of 0.04 h^{-1} to calculate the decay of the constituent.

Figure 5.19 shows the simulated chlorine residual concentrations and corresponding travel times at three different nodes in the network: 108, 155 and 197. The results correspond to the second day of a 48-hour extended period simulation, the first 24 hours being used to stabilise the transport process and provide more realistic initial values for the second day.

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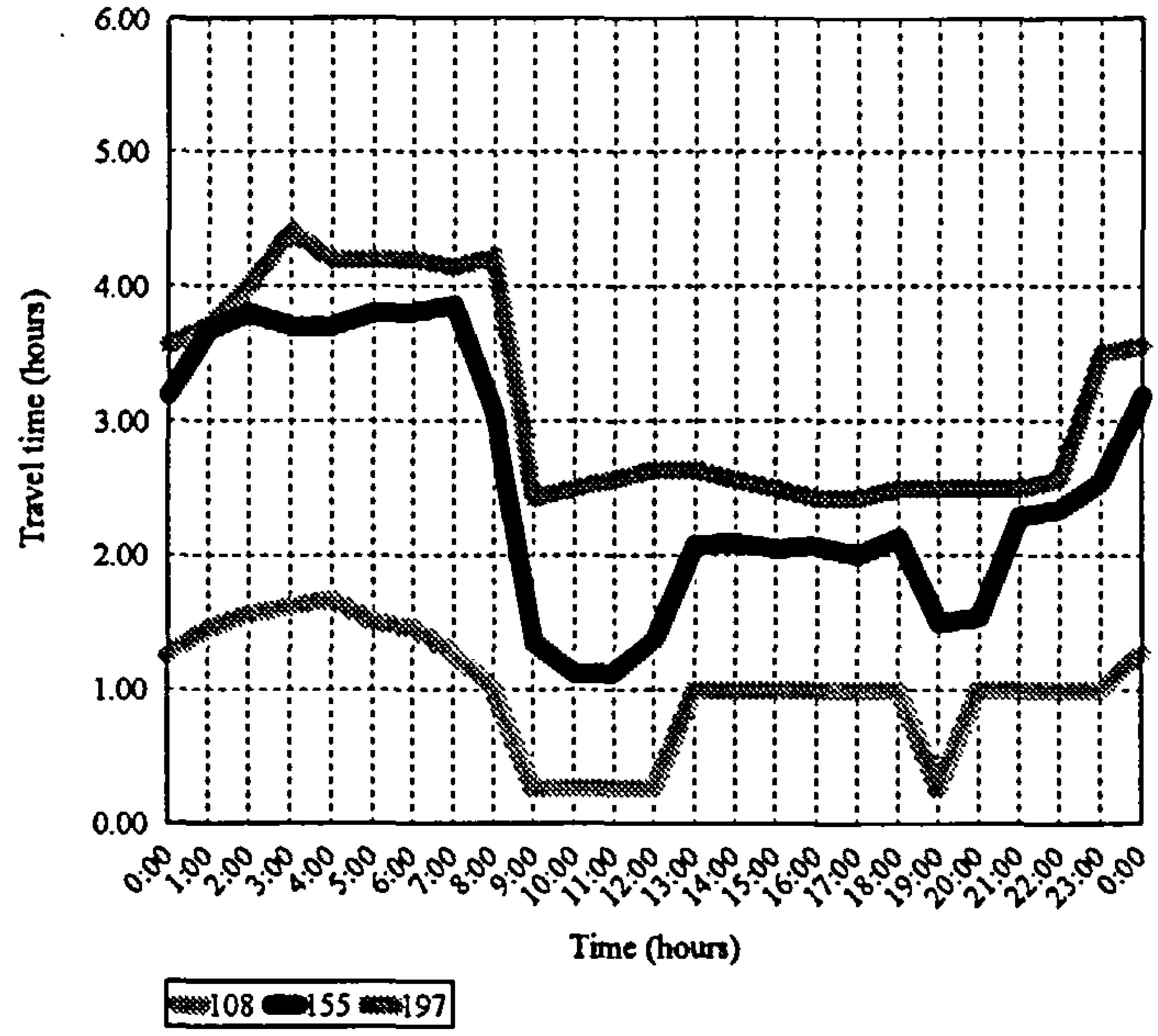
[5] The simulations involving this network, although perfectly realistic from this work's point of view, are performed using a deliberately fictitious distribution of demands and chlorination values and do not therefore represent any actual operational scenario.



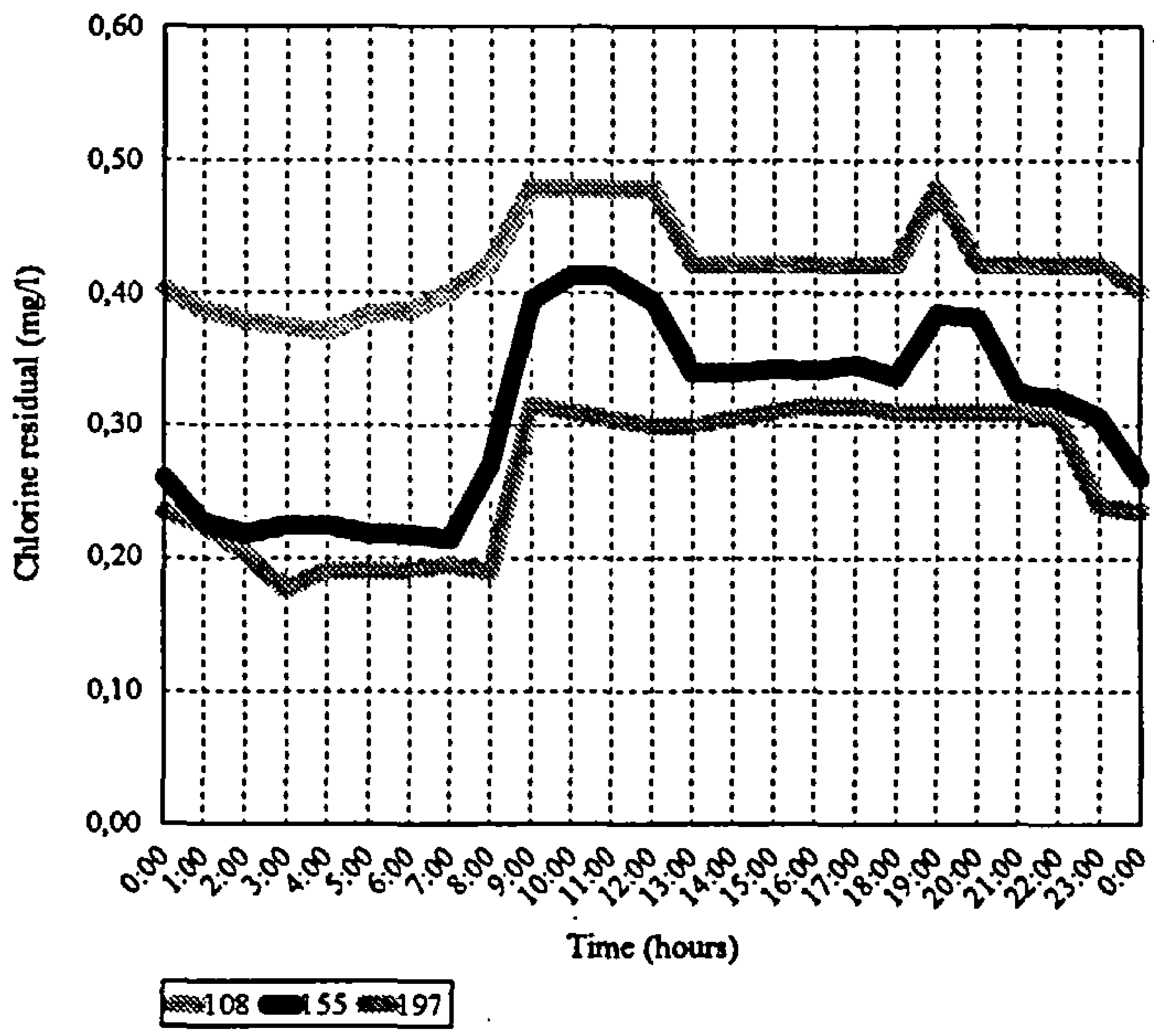


Fig.5.19 -East Edinburgh chlorine residuals and travel time profiles

No field measurements were available for this network to provide comparison with the simulated values, but the graphs nevertheless show the type of result to be expected in a medium-sized, reasonably complex urban distribution system.

A cursory examination of the results shows that some of the chlorine levels at the most peripheral nodes, 155 and 197, are low or insufficient to satisfy the lower recommended limit of 0.20mg/l. That type of problems is to be expected at nodes with longer average travel times. Supposing the hydraulic model is well calibrated, and the transformation function provides a good model, the system would be facing an immediate water quality problem. An increase in the source disinfection rate is undesirable, as it already is on the upper recommended limit. The first step to a solution would be to determine how much would any demand be affected by an increase in the disinfection values. It is possible that there is enough travel time between the source and the first demand nodes to allow for the chlorine residuals to decay down to acceptable values. Failing that, the situation could not probably be improved without resource to booster chlorination nearer the problem areas.

Closer inspection of the results show that most of the low, sub-limit chlorine concentrations occur at night-time, naturally coinciding with the longer travel times caused by the slower flow pattern. During daytime the three nodes behave much better. The plots show that any water quality sampling carried out during daytime will risk missing out on a clear situation of disinfection insufficiency at night. This very simple example nevertheless demonstrates how useful the concentration and travel time results of a water quality model can be in highlighting what would otherwise be less evident problems or shortcomings.

The above assumption of model validity for the sake of highlighting a few simple aspects of water quality simulation is, of course, untested. Nevertheless, the hydraulic model does have a real calibrated base, which allows for some degree of confidence on the advection and mixing processes[6]. But the transformation model must really be questioned from the very start, given the influence of an incorrect k value on such a simple first-order model. An essential procedure is to organise field measurements to at least calibrate the k value as a function of pipe size, one of the most determinant factors in the opinion of, among others, Tansley and

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6 To be fair, it must be said that the degree of skeletonisation of the hydraulic model such as presented may be too high for a water quality model. This is not uncommon in water quality modelling, but is seldom acknowledged.

Brammer (1993), as well as checking the dependency with the range of variation (summer/winter) of the water temperature.

Just to give an idea of how significant is the transformation function in the overall model, the simulation whose results are shown in figure 5.21 was performed using the relationship between *k* and pipe diameter achieved by those authors for a network in England, and shown in figure 5.20. The results, superimposed on the previous ones, illustrate the extent to which a model of the kind shown is sensitive to the simulation of the decay process.

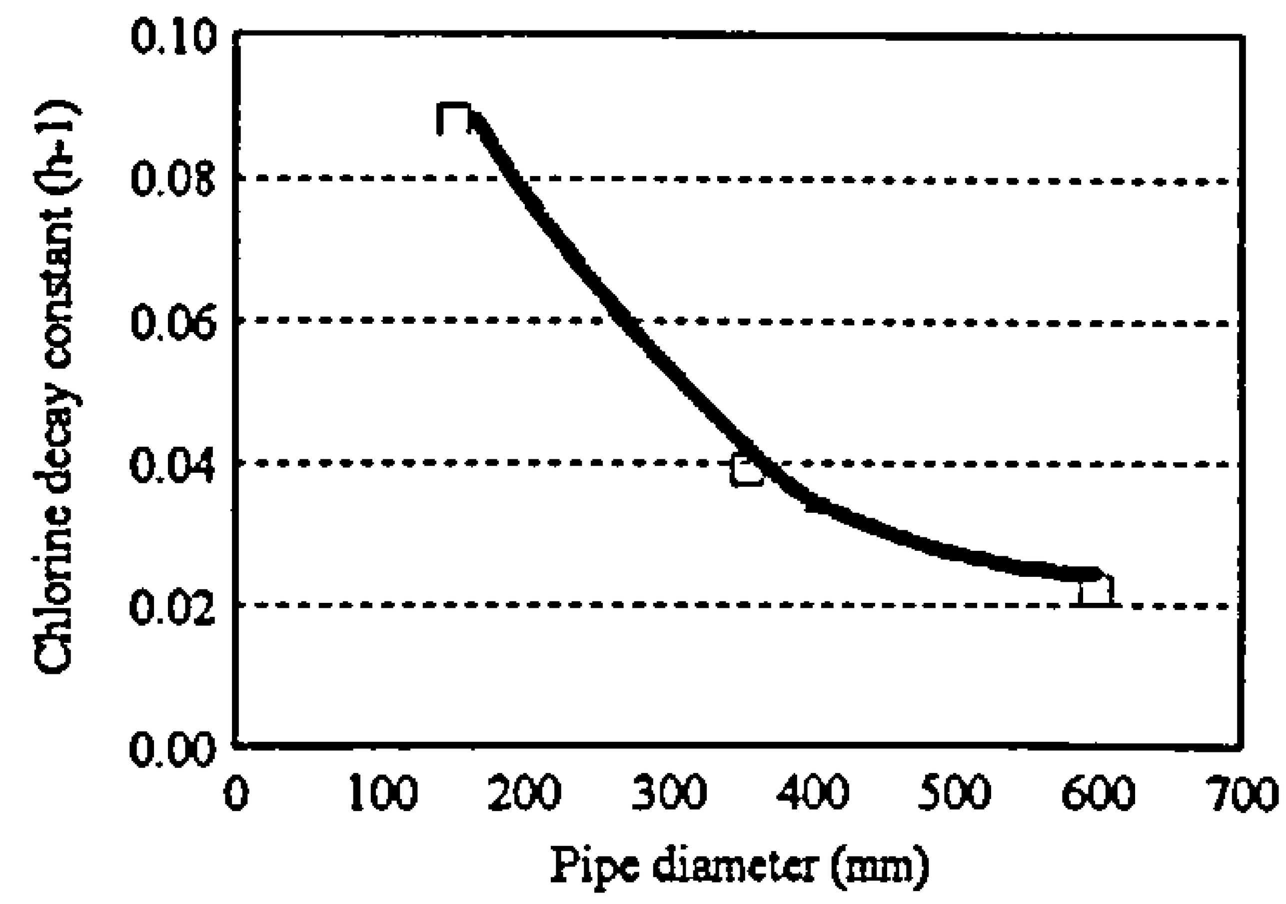

Fig.5.20 - Chlorine decay constant as a function of pipe diameter (Tansley and Brammer, 1993)

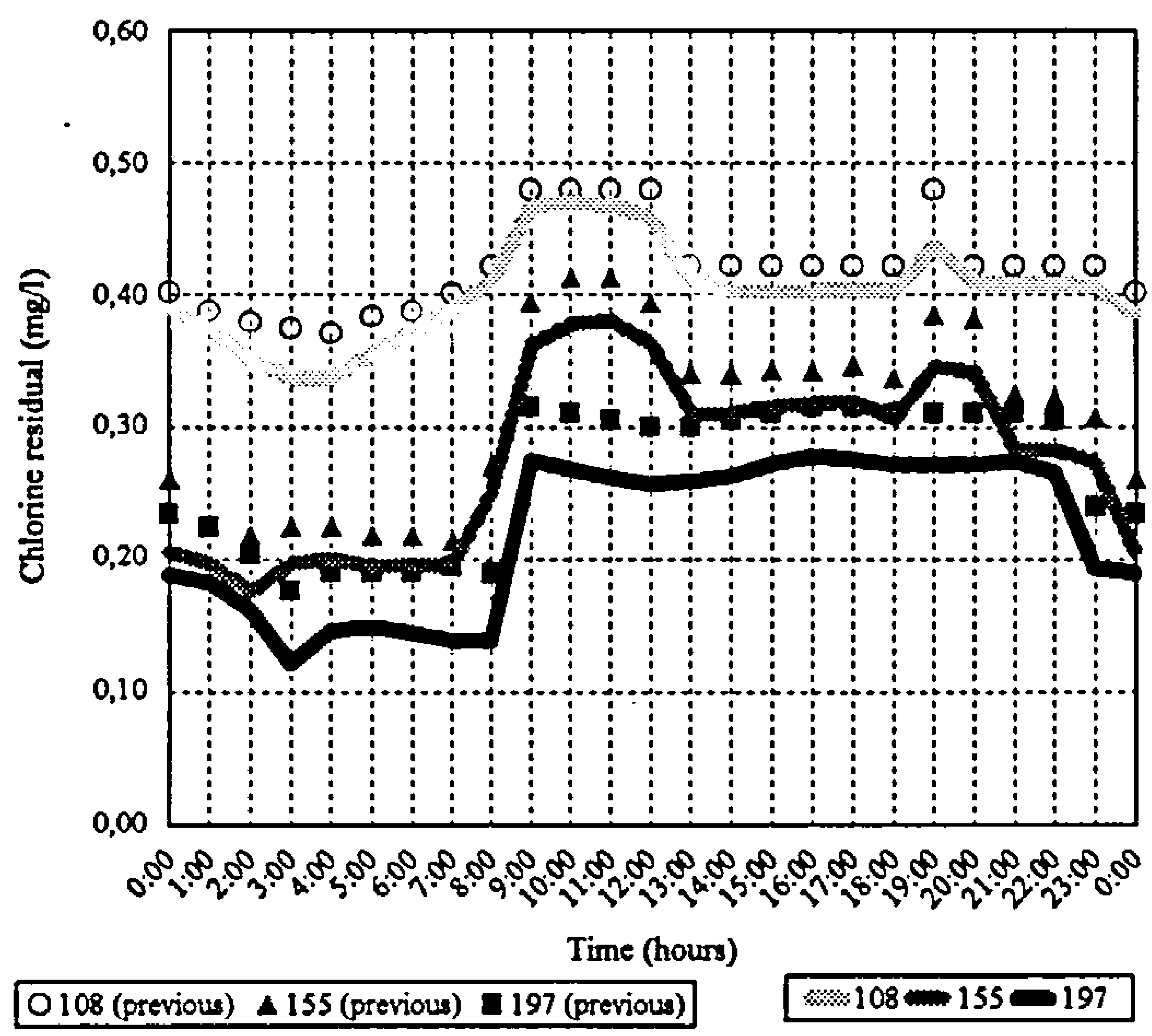

Fig.5.21 -East Edinburgh chlorine residuals profiles for improved *k* values

Case 3 - The SCCRWA EPANET water quality model

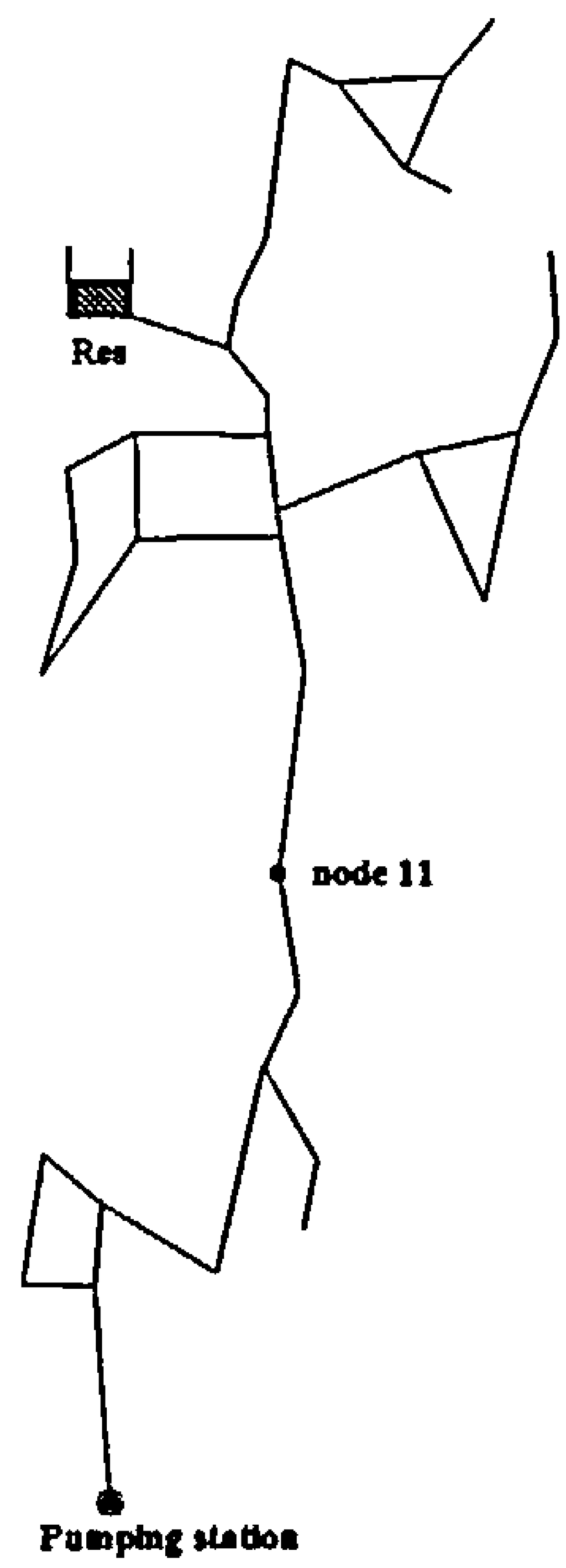

Fig.5.23 - SCCRWA example network

Since it was not possible to obtain field measurements from the East Edinburgh network in order to assess the results from PERFORMANCE-Q, examples of networks with known field measurements available in the literature were utilised for that purpose, with generally good results. One such case, shown in Fig.5.23, is an example network included in the User's Manual of the U. S. Environmental Protection Agency's EPANET package (Rossman, 1994), which effectively corresponds to a real water quality simulation study for a SCCRWA service area (Rossman, 1993).

The network description is included in Appendix A. The field results concern a calibration exercise a using conservative fluoride tracer test. Fluoride addition at the treatment plant feeding the network was turned off and periodic fluoride measurements were taken at several points in the network for the subsequent 55 hours. The exercise was used to actually adjust the nodal demands and calibrate the hydraulic model accordingly. The network data in Appendix A include the final, calibrated nodal demands. Figure 5.22 shows the measurements

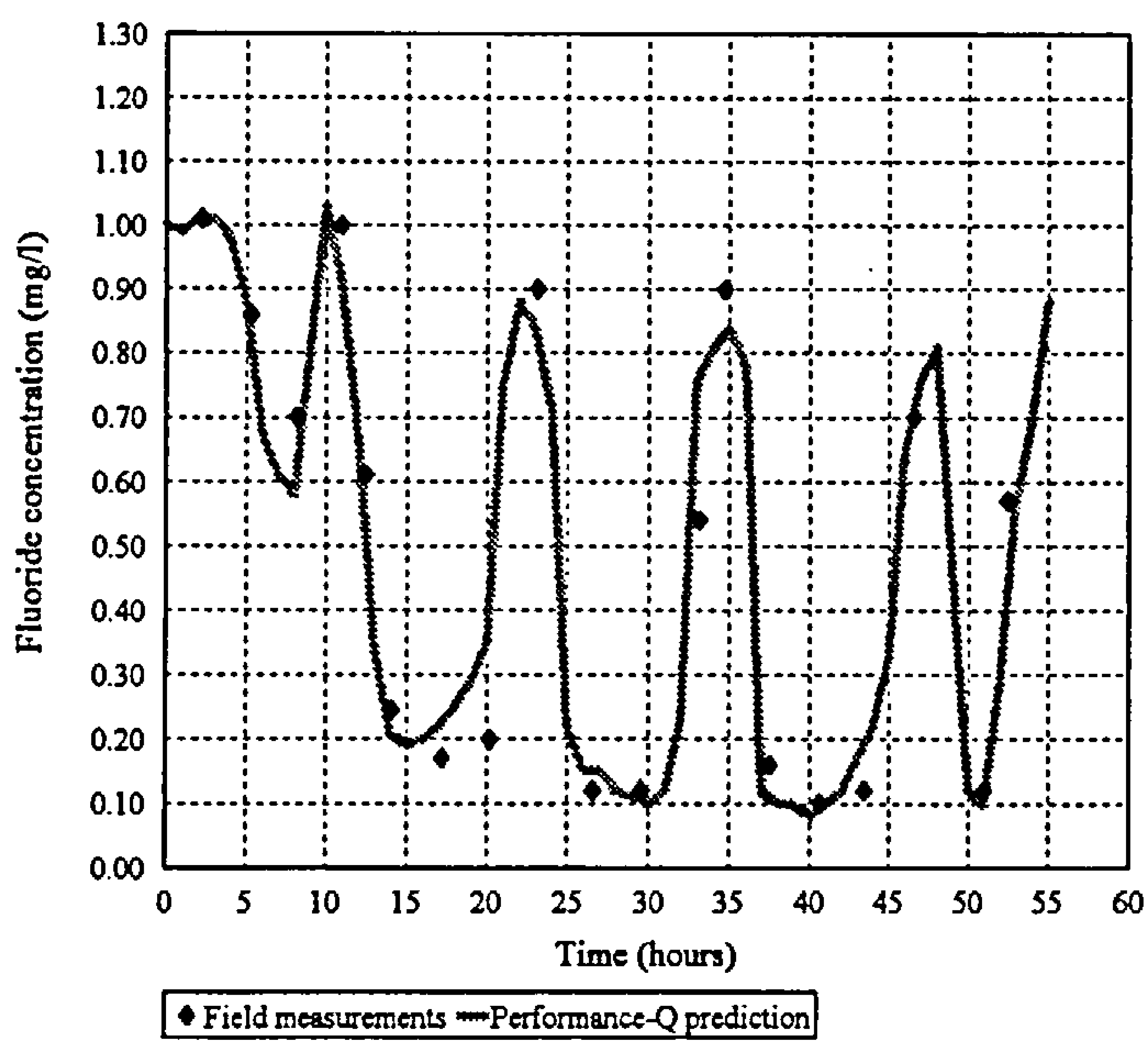

Fig.5.22 - Measured and modelled fluoride levels using PERFORMANCE-Q

taken at one of the nodes (11, as shown) and those simulated by PERFORMANCE-Q using the calibrated model as well. Run times are in the order of 10-15 seconds using a 80486-standard processor running at 75MHz.

5.4. WATER QUALITY PERFORMANCE EVALUATION

5.4.1. Introduction

In similar fashion to previous domains, the main objective of this chapter is to apply the standardised performance assessment framework to the field of chemical and physical water quality in distribution networks. As seen before, the performance evaluation framework establishes three types of entities for each network property or behavioural aspect it analyses: (i) A relevant state variable, that is, the quantity which translates the said property at the *network element* level, from the point of view taken into consideration; (ii) a penalty function, mapping the values of the state variable against a scale of index values; and (iii) a generalising function, used for extending the element-level calculation across the network, producing zonal or global values.

The next sections will deal with those subjects, in particular presenting various possibilities for variables and penalty curves that arise in the water quality field. To place some of the ideas in context, a brief interlude refers to the water quality regulatory framework. After discussing penalty curves and generalising operators, case studies are presented to illustrate the main properties of the methodology.

5.4.2. Drinking water quality standards and regulations

The quality of drinking water has been the subject of increasing standardisation and regulation efforts both at national and international level over the years, having progressively passed from broad requirements for *wholesome* water to a wealth of mostly numerical standards. The main objectives are to guarantee adequate potability and correct mineral balance, but above all, to avoid toxicity or contamination situations.

One of the most important and authorised sources of standardisation are the guidelines provided by the World Health Organisation (WHO), intended as a basis for developing national standards (WHO, 1993a). In the United States, the U.S. Environmental Protection Agency standards are known as Maximum Contaminant Levels (MCLs) and are mainly health-oriented, although considerations such as available treatment technologies and analytical methods are also taken into account. The MCL for a particular chemical is the statutory and enforceable maximum permissible level of contaminant in water which is delivered to any user of a public water system. Secondary, non-enforceable MCLs are listed for inorganic substances which affect the acceptability of the water but are not directly relevant to health. The USEPA also produces lifetime Health Advisories (HAs) which indicate the maximum concentrations of chemicals in drinking water that are not expected, with a margin of failure, to cause any adverse carcinogenic effects over a lifetime of exposure. Comparison between the MCL and the HA of a particular substance provides an idea of how close the standard is to the health limit.

In Europe, the E.C. directives establish the minimum legal framework that the national legislations of the member states must impose. The EC Drinking Water Directive (80/778/EEC) sets up standards which define Maximum Admissible Concentrations (MACs) for those parameters – properties, elements, organisms or substances – which are deemed to be undesirable[7], in a similar philosophy to the MCLs. Conversely, Minimum Required Concentrations (MRCs) are defined for parameters whose presence in the water is essential. For both cases, further to defining the extreme value, the standards also define Guide Values (GV) as the actual goals for the water quality parameters, taking in consideration all sorts of factors – such as aesthetic aspect, long-term exposure (in similar fashion to the American HAs), effect on the distribution system, combined effects, etc. – and not just the strictly health-based criteria.

In Britain, the 1989 Water Act and the 1991 Water Industry Act (and their Scottish equivalents) incorporate all the standards set out in the EC Drinking Water Directive, as well as 11 further national standards (Drinking Water Inspectorate, 1993). The standards are defined in terms of Prescribed Concentrations or Values (PCVs), which are the numerical values assigned to the maximal or minimal legal concentrations or values of water quality parameters, in other words to MACs or MRCs. The PCV may in certain circumstances be authorised by the Secretary of State to be relaxed to a specific extent, to account for emergencies, exceptional or particularly unfavourable conditions, and subject in most cases to the completion of improvement works. This in a sense incorporates a bit of the Guide Value philosophy.

In Britain, a total of 55 drinking water parameters are regulated by numerical standards, used in 3- or 12-month averages, while another two are the subject of descriptive standards. In Britain, a list of those parameters with their PCV's can be obtained from the Drinking Water Inspectorate (see DWI, 1993). Further standards apply to water leaving the treatment works and in storage reservoirs.

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7 The term is intended in a general sense, therefore encompassing any toxic, contaminant or otherwise unwanted substance in drinking water.

5.4.3. Variables and penalty curves

The preceding brief overview of the water quality regulatory framework aims at introducing the requirements that can be most directly associated with the performance of a water distribution network. The objectives that drive a performance analysis in the water quality camp can be as diverse and wide-ranging as in any of the other domains. Tansley and Brammer (1994), summarise a water distributor's typical point of view: *"[The company] is committed to both satisfying the demands of the regulatory bodies, and providing its customers with the best level of service at the lowest possible cost. (...) It is recognised that although water may comply with all the Prescribed Concentration or Value (PCV) standards, the optimisation of taste, odour or appearance of potable water is essential to maintain and improve service levels and the image of the company. Indeed, recent surveys have suggested that the customer is willing to pay extra for active efforts to be made to make water not only safe, but pleasant to drink."*

The statements define in a nutshell what is very much the function of a water utility. The level of service is understood as not only the compliance with the accepted industry practice and the regulatory framework, implicitly incorporating all the identified quality requirements for a proper drinking water, but also, and quite importantly, the provision of a *pleasant* product. In other words, something that the customer can see and perceive as good quality, hence good service.

Of the various applications of water quality models such as described in the previous sections, it is clearly the simulation of concentrations and travel times that has the potential to fit into the defined performance assessment framework. Both are nodal-style variables, as produced by the model.

In the first case, the regulatory framework provides an ideal platform on which to elaborate. Most of the prescribed standards for drinking water parameters, being numerically defined, are just adequate for the task. The parameters they refer to fulfil all the requirements outlined in Chapter 3 for the state variables or properties, and the very definition of the standards is a good starting point for laying out penalty curves.

In the most basic and typical situation, the water manager or designer will want to ensure that a given parameter, or set of parameters, will comply with the respective Prescribed Concentration or Value. Curve (a) in Figure 5.24 shows a possible penalty curve for an unwanted substance. The optimum level of service (4) will correspond to all values up to and including the PCV, above which it will simply drop to zero. Theoretically this should be a sudden drop, if the definition of a Prescribed Concentration or Value is to be taken literally. However, the situation will arise where either there is case for an official relaxation value (Rv), or quite simply the technical manager is willing to introduce his own "relaxation" as a sensitivity gain manoeuvre for analysis and diagnostic purposes. In fact, an all-or-nothing type of law is often much less informative than one that can tell us something about the degree

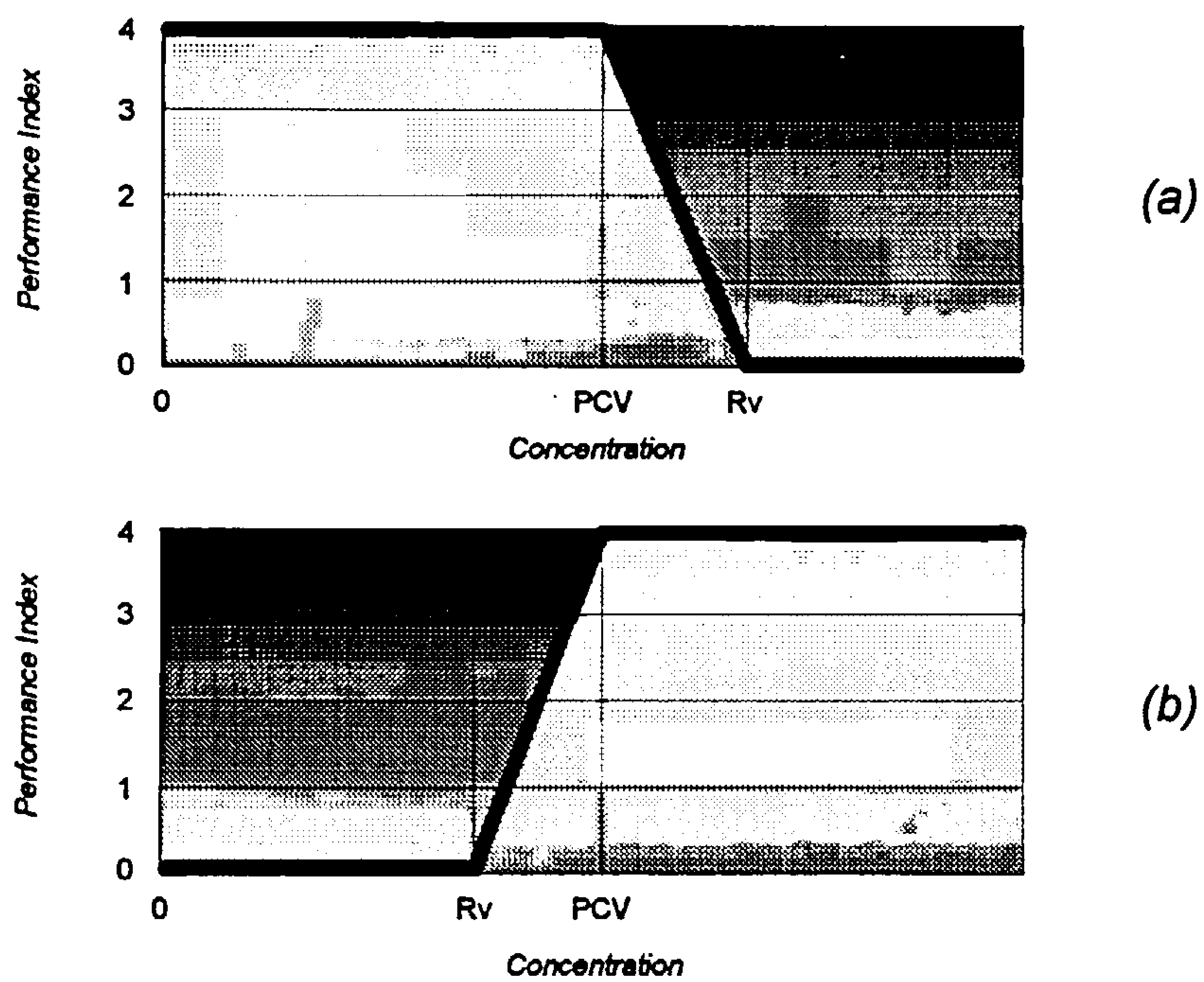


Fig.5.24 - Penalty curves for (a) unwanted substance and (b) essential parameter, using PCV

of failure, as is the case if a gradual or even steep decrease is introduced.

This issue is not totally unconnected, at least from a conceptual viewpoint, to the way in which the regulations are expressed as regards sampling frequencies and the definition of a good water. Many regulations, and the E.C. directives are an example, rely on deterministic criteria that either impose a total compliance with the standard or are not precise on how to interpret it. Others begin to recognise the impossibility of guaranteeing a total compliance in the whole of the supply and distribution system using the prescribed sampling procedures and use a more statistically-based approach that realistically defines small tolerances for the number of violating samples.

In any event, the penalty curves are always viewed here as first and foremost a technical management decision-support tool. Their shape will therefore tend more to reflect the view of the analyst for the particular purpose, scenario or simulation in view, even if constrained by the regulatory directives, than to merely translate those.

In the same figure, case (b) refers to a parameter whose presence in the water is desirable (such as fluoride) or essential (such as a disinfectant agent). In this case, we have a reciprocal situation to (a). The difference between desirable and essential can precisely correspond to having a slope as in the figure, or a vertical drop below the PCV.

A Guide Value logic will now be employed to make the curves slightly more elaborate. Figure 5.25 shows a possible configuration. The optimum situation is now really the GV, to which a value of 4 is assigned. The MAC or MRC are at the adequacy limit (2), and beyond them it is a no-service situation (0).

On the other side of the GV, it is a lot less clear which way to go, since it depends very much on the particular parameter and on the nature of the GV. Often in the (a) case, achieving concentrations much below the GV increases the treatment costs so much that it becomes

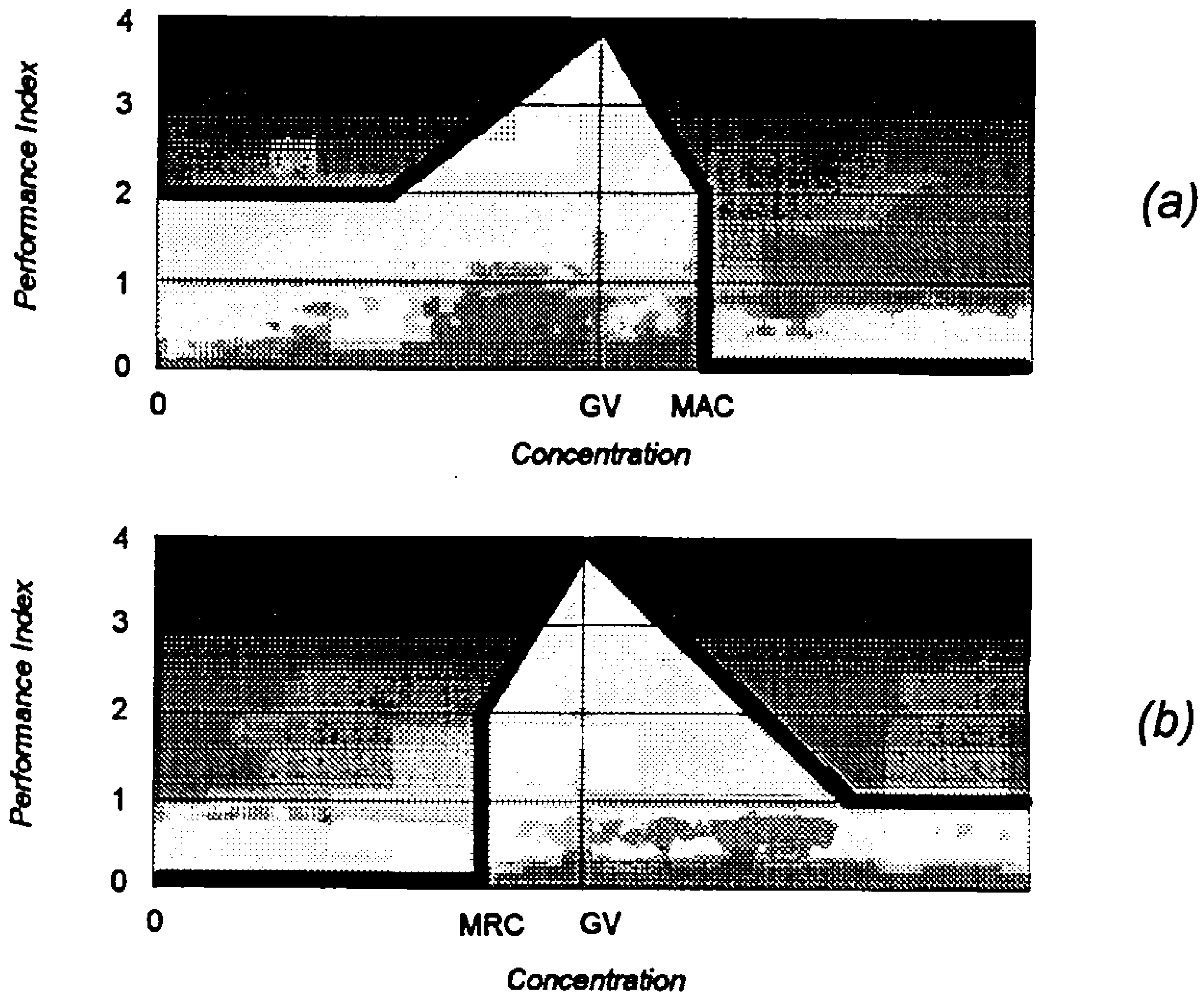
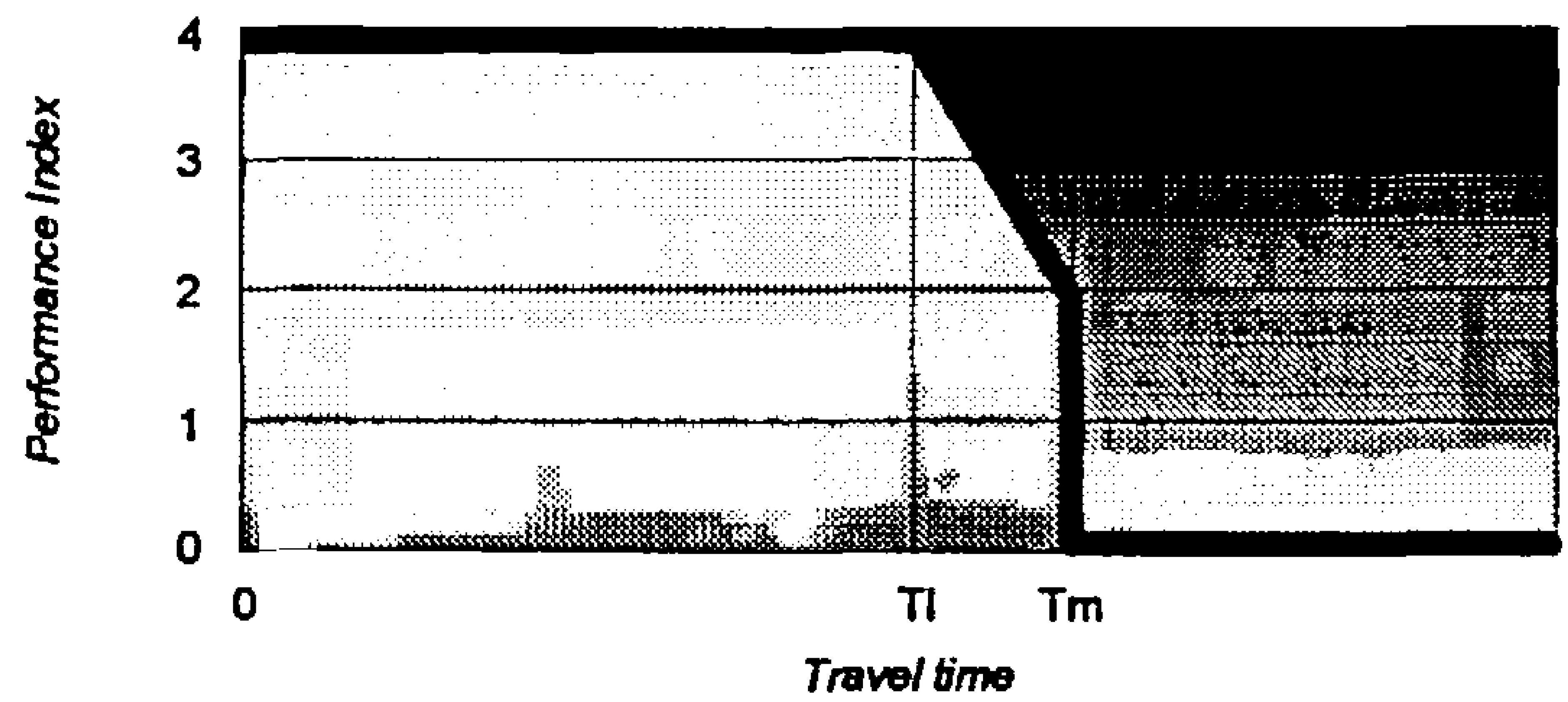


Fig.5.25 - Penalty curves for (a) unwanted substance and (b) essential parameter, using GV

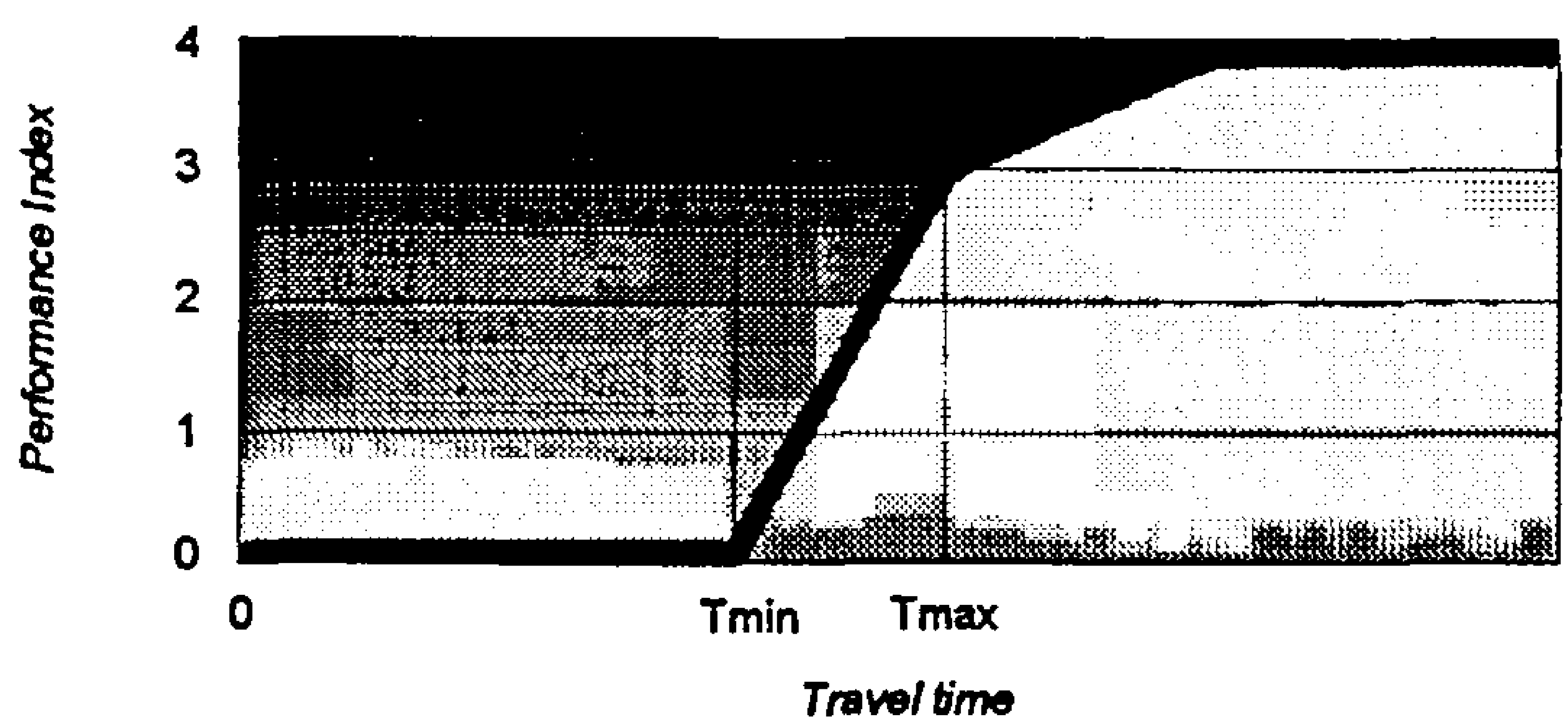
undesirable, hence the arbitrary drop shown in the figure to merely acceptable, at 2. This would be an example of a multi-constrained penalty curve, looking at both quality and cost. In actual fact, the Guide Value happens to be simply zero for many undesirable substances, which solves the problem. In the (b) case, values above the GV may often correspond to increased treatment costs or imply undesirable side effects. A good example of that is chlorine residual, which will be illustrated in later sections.

Travel times are the other product of water quality models that lends itself to a straightforward performance assessment treatment. Again a nodal variable, its calculation is normally spurred by a concern with the age of water, a simple indicator of the overall quality. From that point of view, it can become a very useful tool in that it can combine a global assessment of the quality of the water in a synthetic single value. For a particular type of water, it is easier to know that it withstands well a given travel time than to calculate and compare a variety of the different components.

Figure 5.26 shows penalty curves corresponding to two different cases of application of travel time as performance assessment variable. The first is the straightforward water age case, where a certain time limit will be defined and the network will be tested for compliance with that limit. The penalty curve is a very simple one, grading any travel time below the limit (*Tl*) as optimum and allowing a certain tolerance above that, defined through a maximum time (*Tm*), to drop down to the acceptable level. Above that, it is a no-service situation.



(a)



(b)

Fig.5.26 - Penalty curves for (a) water age and (b) reaction to incident

The second case, curve (b), refers to a much more specific situation which illustrates a very "topological" application of travel times, that of testing the effect of a contamination incident. In this case, travel time is graded against the response capabilities of the water utility. The longer the travel time to a particular node, the more likely a consumer supplied by it is of being warned in time.

Let *Tmin* be the shortest time in which the company can begin issuing warnings, and *Tmax* the time elapsed before all its customers can be guaranteed of being warned. The resulting curve shows a "safe" (good) grade above *Tmax,* becoming "very safe" (optimum) well above *Tmax,* an "unsafe" (no-service) grade below *Tmin,* and a varying degree of concern (level of service) between *Tmin* and *Tmax.*

5.4.4. Generalising functions

What was said with respect to the deterministic or statistical basis for the application of the drinking water quality standards is a good introduction to the subject of generalising functions. The most natural generalising operator for the kind of penalty curves exemplified above, which are constrained by the regulatory approach, is undoubtedly a 0% or very low percentile of the nodal index population. Using the 0% percentile or, in other words, the very worst nodal value as representative of the network, is to a certain extent mirroring the *total* compliance with a standard. Using the very low percentile, say 5%, will allow for a certain margin to accommodate the few worse cases.

Apart from the 0% value, whatever percentile is calculated, be it as generalising operator or for drawing the dispersion bands, will raise the question of whether to or not to weight the nodal indices as in previous cases. Again as before, it makes sense to use the nodal demands as weights, since larger flows of an incorrect water will affect more people, be more expensive to correct and generally be more representative of a problem than smaller ones. The percentiles will therefore correspond not to the number of nodes but to the percentage of total demand with an index below or equal to the Y-coordinate of the corresponding point in the appropriate curve in the graph.

5.4.5. A note on system curves

The simulation of the transport, mixing and transformation water quality parameters is very much travel-time dependent, and therefore has a sort of *sequential* nature. The influence of the diurnal variability of demands and flows on the mixing patterns and travel-times is believed to be crucial for the validity of the simulation. For that reason, the production of system curves as defined in Chapter 3 appears not to be applicable to this case. Quite apart from the fact that the automated system simulation described before would have to be replaced by a more cumbersome procedure in which each demand factor would have to be simulated individually in steady state for a period of time long enough for the transport, mixing and decay processes to stabilise, the curves obtained by uniting those points in the graph would hardly have the same significance as for the hydraulic measures. The latter are near-instantaneous properties that can actually and realistically vary from one value to the next in the system simulation curve. It would not be the case if water quality system curves were drawn.

For that reason, the performance assessment of water quality parameters will be based solely on extended-period simulation curves.

5.4.6. Application examples

The East Edinburgh distribution system is again used here to illustrate the most typical applications of the penalty curves discussed previously. The examples concern a performance analysis of the disinfection by chlorine analysed in Case 2 of 5.3.7., always bearing in mind that the simulated scenarios are fictitious.

The WHO recommends that a free chlorine residual of 0.20 to 0.50 mg/l be observed throughout the entire water distribution system. The daily maintenance and monitoring of a chlorine residual offers two benefits. Not only does the residual suppress the growth of organisms within the system and protects against contaminants entering it, but also its sudden disappearance provides an immediate indication of the entry of oxidisable matter into the network or of a malfunction of the treatment process. Booster or relay chlorination may be needed to ensure that this residual is maintained throughout the system.

Conversely, it is recognised that excessive levels of free chlorine may react with any organic matter to produce unwanted by-products such as carcinogenic trihalomethanes, as well as taste and odour in some waters (WHO, 1993b), hence the upper limit recommendation. Even though the occurrence of unfavourable organoleptic characteristics in the water may be due to a variety of parameters, the effects of excess chlorination, either by itself (chlorinous smell and taste) or as the product of its reactions within the water network environment are very often the source of complaints. The current WHO guidelines define the range 0.6-1.0 mg/l as the sort of values that generally begin to cause problems with acceptability. The value of 5 mg/l is, on the other hand, the health-based guideline, defined as the maximum advisable concentration if 100% of the total daily intake (TDI) is allocated to drinking water.

The optimisation of taste, odour and appearance of drinking water is essential for the perception of a good quality service by the customer of a water utility. The network manager has therefore to face the problem of how to balance the two conflicting objectives:

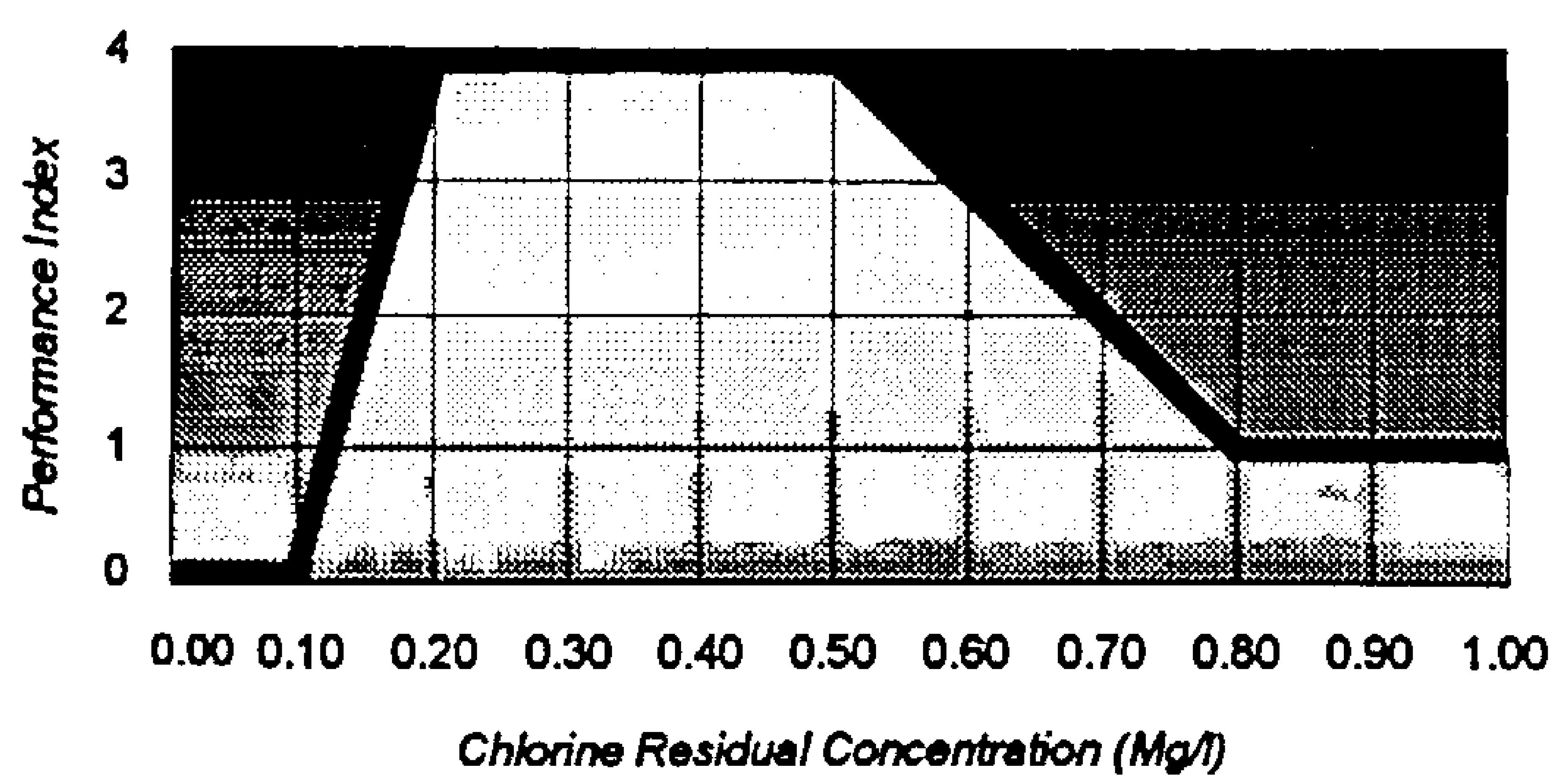


Fig.5.27 - Penalty curve for chlorine residuals concentration

guaranteeing adequate disinfection levels while minimising the presence of excess disinfectant and the aesthetic nuisance it represents.

Figure 5.27 shows a plausible penalty curve reflecting such a viewpoint. It is similar to 5.25(b), with a slightly more tolerant acceptance of the values outside the recommendation guidelines, mainly to avoid sudden discontinuities that may make it more difficult to interpret the results. The optimum range is defined between 0.20 mg/l and 0.50 mg/l, the essential range that establishes the disinfection function. A tolerance is given above the upper limit, but the performance nevertheless drops due to those organoleptic effects: 0.60 mg/l is graded "good" and 0.70 mg/l is though to be at the threshold of acceptability, while beyond 0.80 mg/l the service is considered unacceptable. On the other side of the lower limit, a tolerance is also allowed but is much tighter given the public health implications of insufficient disinfection. A chlorine residual concentration is no longer acceptable below 0.15 mg/l, and anything below 0.10 mg/l effectively corresponds to no service.

Applying the penalty score thus defined to the scenario simulated in the Example Case 2 of 5.3.7. produces the extended-period performance simulation shown in figure 5.28. This time a disinfectant source concentration of 0.70 mg/l is used in order to improve the overall situation of the peripheral nodes, following the suggestion made in 5.3.7. of exploring an increase in the overall disinfection. The simulation is performed with the improved transformation model using the pipe size-dependent chlorine decay constants from figure 5.20.

The network seems to behave mostly well, with only part of the lower percentile displaying night-time problems, due to increased travel times caused by the lower velocities of the night flow pattern. The corresponding water age performance simulation is plotted in figure 5.29, using a penalty curve of the type shown in 5.26(b), with *Tl* and *Tm* respectively 6 hours and 10 hours. The travel times plot confirms the timing and significance of the nocturnal drop, in spite of the two penalty curves being only very roughly calibrated to one another.

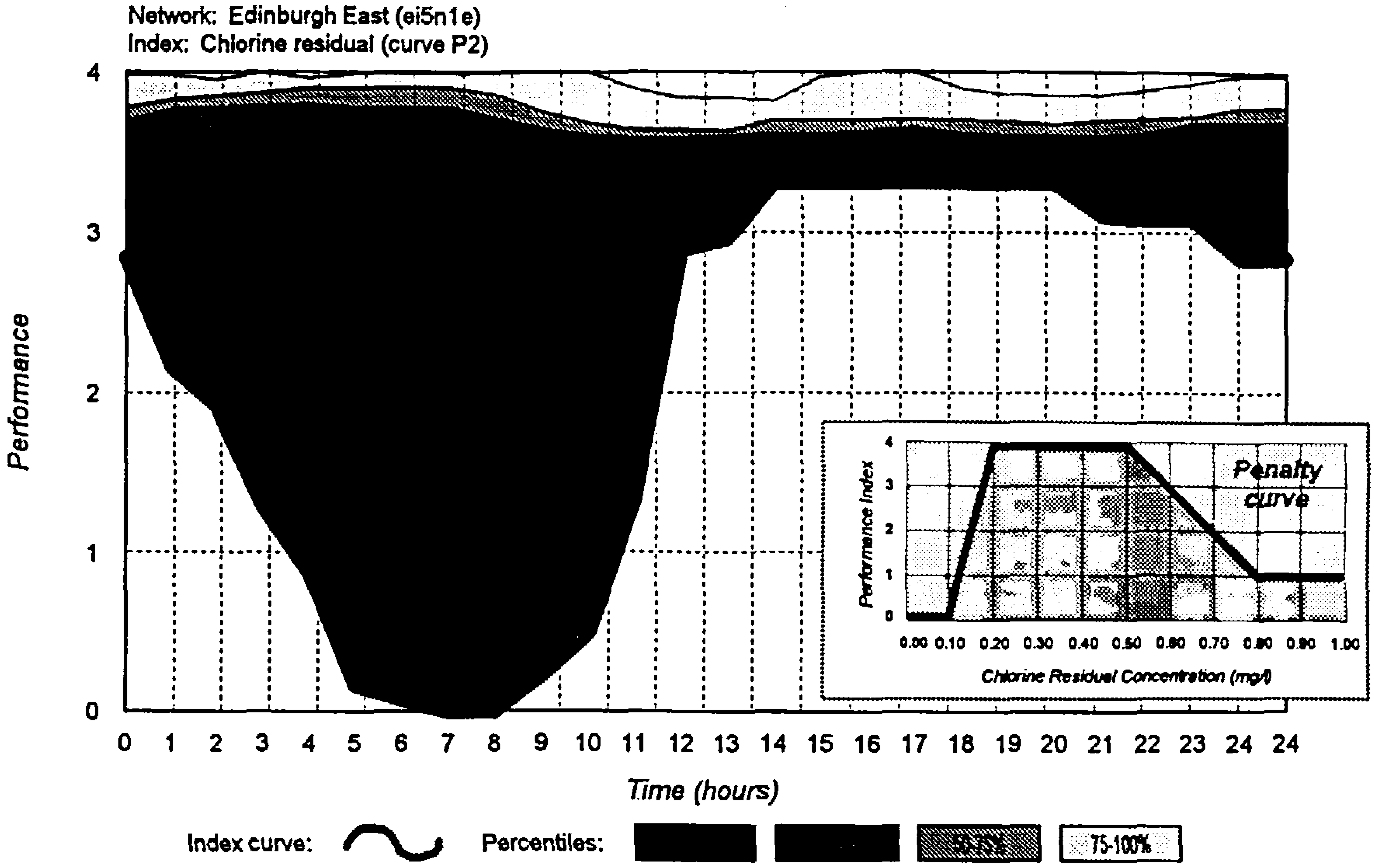


Fig.5.28 - Extended-period performance simulation for chlorine residuals

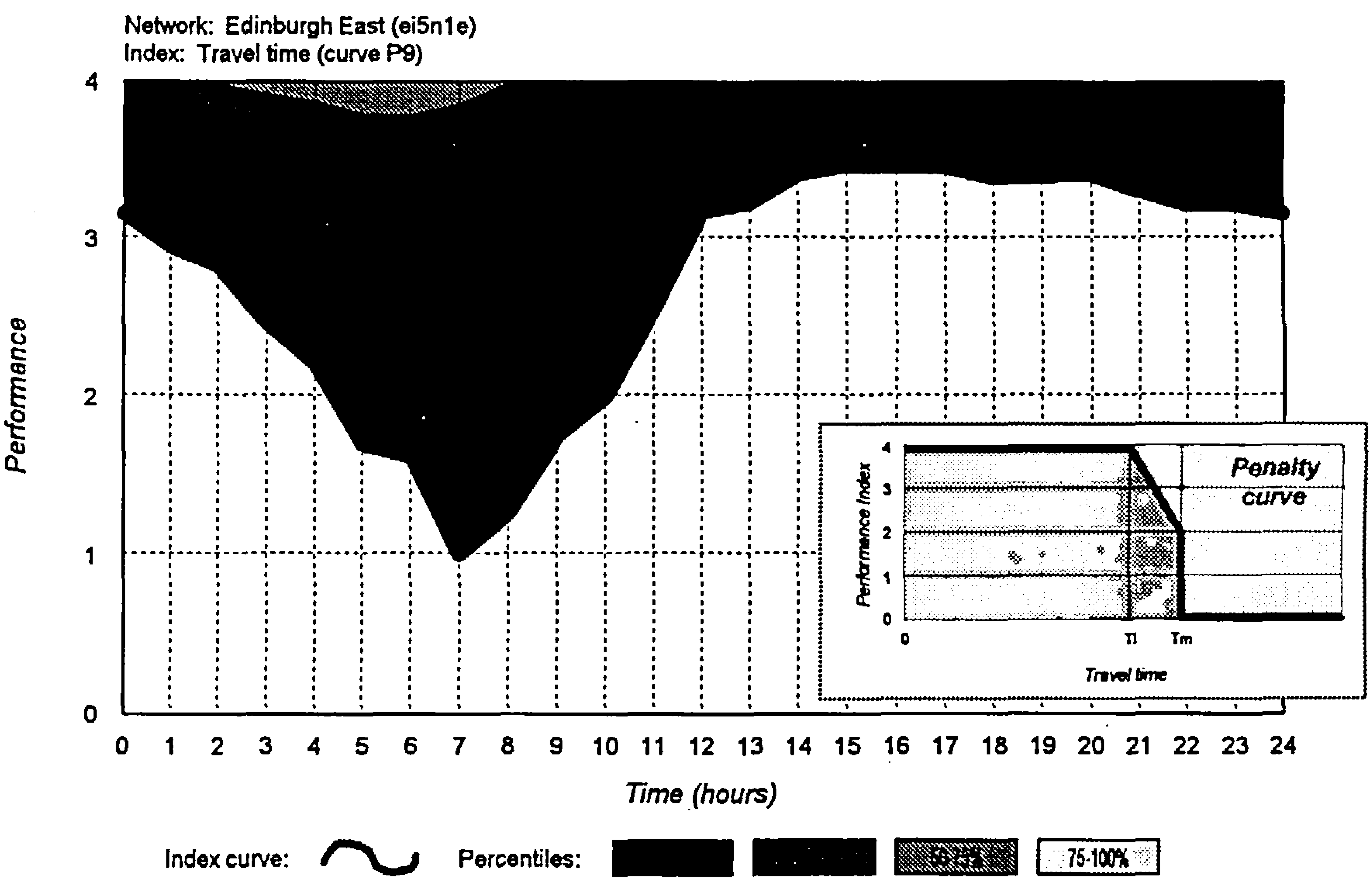


Fig.5.29 - Extended-period performance simulation for water age

With no concentration values above 0.70, it seems clear that the lower performance at night is caused mainly by sub-0.20 mg/l, insufficient disinfection problems. Any effect due to excess chlorination in areas near the source would tend to become more visible at peak flow times, when the velocities are higher and the resulting faster advection will make those high concentrations reach a greater number of nodes.

However, during peak hours (12:00 to 20:00) the diagram is rather narrow, with the whole system performing quite well above a 3.25 index value, which indicates that no demand node gets more than about 0.575 mg/l [8].

The graphs lead us therefore to conclude, that, for the analysed hypothetical scenario:

- the network would have areas with insufficient disinfection problems[9], probably corresponding to the peripheral nodes with longer travel times of what is effectively a rather elongated-shape network; and

- there would be no significant organoleptic concern, given that only a small percentage of the demand would catch the full force of the excess chlorination at the source.

To try and improve the performance of this scenario, a simulation with 0.75 mg/l at the source was carried out. Figure 5.30 shows the corresponding performance plot. There is an overall improvement to the night-time situation, together with a compression of the top 25% band further away from the optimum levels that seems to confirm the earlier interpretation of the previous diagram. For this network and the penalty curve used, the disinfection strategy thus defined seems to provide now an overall good performance.

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[8] Again some caution should be observed with this type of conclusion, which is taken here as merely illustrative. In the particular case of this network, the excessive aggregation of demands for water quality modelling purposes can, and probably does, bias the results to some extent.

[9] It is reminded that the present illustrative examples, and the Example Case 2 of 5.3.7. upon which they are based, are totally fictitious. Although realistic, they must not be taken as corresponding to any existing or predictable situation at the East Edinburgh distribution network.

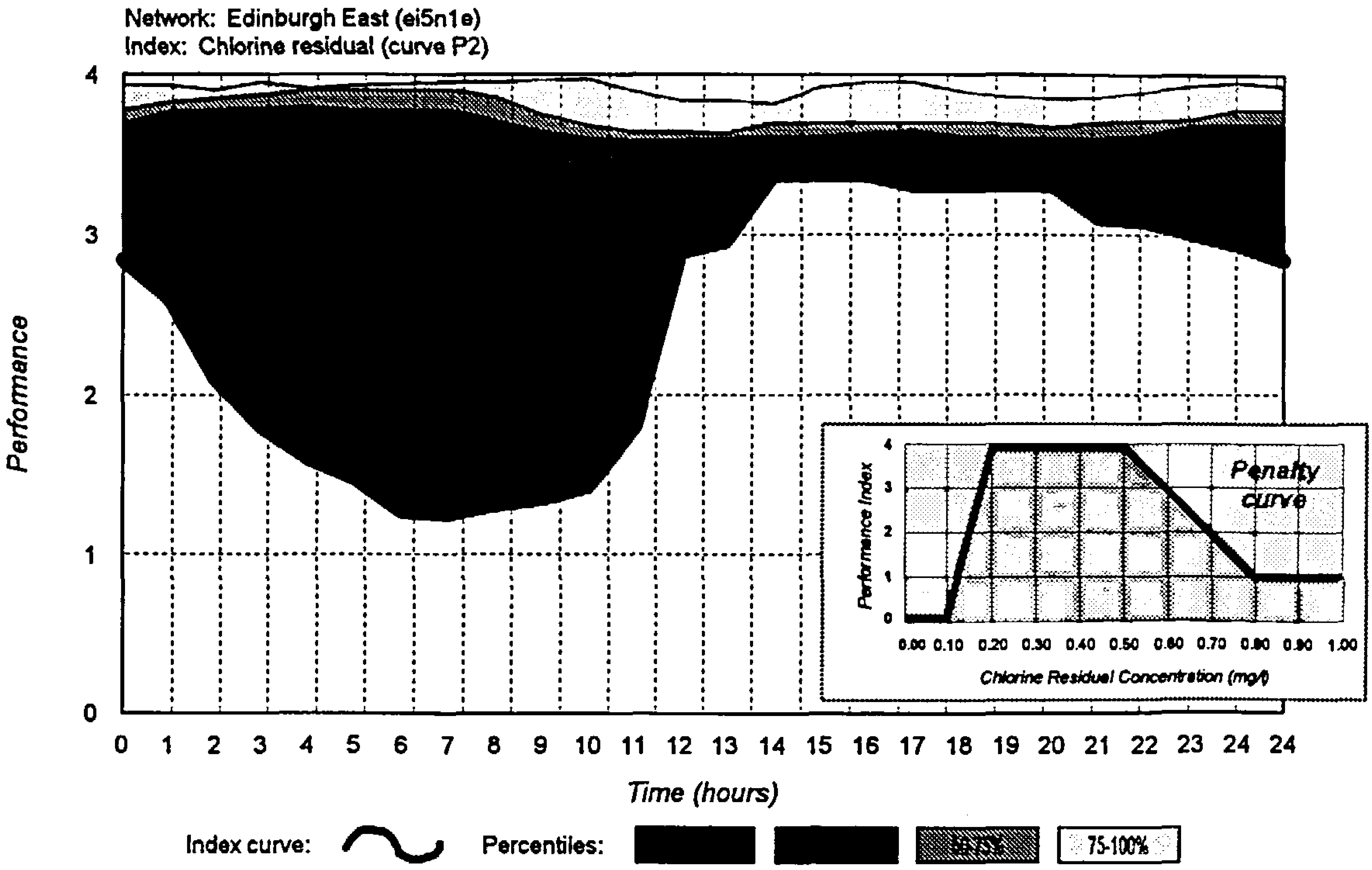

Fig.5.30 - Extended-period performance simulation for chlorine residuals with improved disinfection

5.5. SUMMARY AND CONCLUSIONS

The present chapter applies the standard performance assessment framework to the field of water quality in distribution networks. A review of the most important issues regarding the control of water quality is carried out before focusing on the subject of water quality modelling as a prime tool for providing a basis for the performance evaluation programme.

Given the need to make use of one such model, not readily available from other sources, and the possibility of exploring an improved methodology, an accurate multi-parameter dynamic water quality model for physical and chemical parameter simulation was developed and described in detail in the present chapter. The method comprises the simulation of parameter concentration, travel time and source contribution across the network and storage tanks and is illustrated with the help of suitable examples. It is argued that the model developed, implemented through computer programme PERFORMANCE-Q, is a robust, efficient and, above all, numerically sound method which does not intrinsically introduce any numerical diffusion when modelling the advection and mixing processes. In this respect it is thought to hold its advantages over some of the documented, established methods.

The method is based on tracing the history of events exactly as they happen, from the sources and downstream across the network. It is computationally economical, holding at any one time no more than the actual number of different concentration segments thought to be present in the system. Its flexibility is also important, with various alternative methods of segment aggregation schemes, various transformation functions, the possibility of simulating an array of parameters simultaneously, and travel time and source contribution calculations as well as parameter concentrations.

The performance assessment methodology is followed through the selection of two principal decision variables – parameter concentrations and travel times – for which penalty curves and generalising functions are developed taking into consideration a review of the regulatory aspects in drinking water quality as well as the more diverse operational and technical management policy objectives. Previous water quality examples are completed with the performance evaluation procedure for cases of disinfection improvement and response to contaminant incident, illustrating several of the main properties of the method as applied to water quality.

Having been integrated with the performance assessment methodology, PERFORMANCE-Q constitutes an innovative first proposal for a performance-driven water quality model. The change in philosophy from the traditional approach yields some considerable benefits for the water systems' engineer, not only enabling a direct, performance-oriented analysis that can be easily standardised to afford non-specialists the best informed views of the problem, but also accelerating the process of sensitivity analysis and the gain of knowledge over the system and its behaviour.

CHAPTER 6

RELIABILITY OF WATER DISTRIBUTION SYSTEMS

6.1. INTRODUCTION

Water is a commodity essential to human life, and a supply and distribution system a fundamental utility in urban agglomerations — a lifeline whose importance is probably only fully realised when it breaks down or in some way fails to deliver. For that reason, urban water distribution networks are designed to supply water for domestic and industrial purposes 24 hours a day, every day of the year. Any interruption to that service lasting more than a few hours will have a rather undesirable effect on the consumers and their activities. Further to the costs involved, it is normally understood in well developed communities that such interruptions are unacceptable: the supply should not only be available in the best possible conditions, it should also be reliable. In the words of the International Water Supply Association (IWSA, 1995): "The community has the responsibility to ensure that water of unquestionable quality, in sufficient quantity, with enough pressure and at a price that covers its costs is at all times available"

Since the random failure of network components is inevitable, water supply and distribution systems should to a certain extent incorporate design and operative measures that make them less vulnerable to such failures. Adequate water supply should in fact be guaranteed under a random failure of one or more network components.

The preceding two chapters analysed some of the most palpable aspects, both for the designer or technical manager and for the consumer, of the performance of a water distribution system. Generally speaking, those aspects translate the very objective of a water utility: to satisfy all demands with sufficient and wholesome water, at adequate pressure, and at the minimum possible cost.

Further to finding out to what degree that objective is accomplished, the performance of a water supply and distribution system can also be measured by how consistently or *reliably* it actually does so. Indeed, some authors associate the idea of performance of a water network mainly to its reliability (Hashimoto *et al.*, 1982; Mays, 1993; etc.). In the framework of the present study, performance of a water supply and distribution does have a broader meaning, but must certainly include some measure of the reliability of the system.

Reliability of a network can also be said to be a *potential* performance measure, as opposed to the *operational* characteristics of the two other fields covered. That is to say, reliability is more clearly introduced and controlled from the designer's desk as a potential feature of the network, than at the operational stage. It is more of a property of the network than an everyday *modus operandi*. One of the reasons for that is the crucial role of the network layout in the level of reliability that it can possibly offer. However, the layout is also the feature of a built network an engineer can do less about. It will be seen later that some approaches to reliability also consider it from the point of view of day-to-day operational scenarios.

It has already been mentioned that Hashimoto *et al.* (1982) define the following concepts as part of their performance assessment for a water resource system: i) *Reliability*, or how often the system fails; ii) *resilience*, or how quickly the system recovers from failure; iii) *vulnerability*, or how serious the consequences of the failure may be. Bouchart, quoted by Goulter (1987), provides some variations on those definitions, based on similar concepts from the field of stochastic water resources. Resilience is characterised by the ability of a distribution system to supply demands in times of component failure; vulnerability is the maximum deficit in supply in terms of network failure; and finally, reliability is given both by the number of component failures in a given time period and by the flow capacity of the (presumably reduced) system.

This chapter will attempt to select and develop a performance measure of reliability in accordance with the essential requirements earlier established as the basis for a performance evaluation framework. In contrast with the previous two chapters, where it was relatively straightforward to identify at least some relevant performance aspects and the properties or state variables that translated them, being more the case of reaching a satisfactory or efficient method of their calculation, here it is less clear what exactly the analysis is attempting to measure, let alone finding the property or variable that translates it.

The next section reviews some of the most important methods for reliability evaluation mentioned in the literature. The review divides the available techniques into direct and indirect methods, and discusses how the concepts of reliability and redundancy can be associated and how the latter may be better evaluated using indirect techniques.

The use of maximum entropy flows is one of the main methods for indirect or surrogate evaluation of reliability, and provides the basis for the reliability performance evaluation proposed in this work. The third section begins by briefly introducing the most relevant points of the entropy maximisation methodology. The evolution of entropy maximisation applications to reliability assessment in water distribution networks is traced and the methodologies so far available in the literature commented upon. A new formulation is then proposed which corrects or completes some of the published methods, followed by a discussion on the suitability of entropy maximisation for reliability evaluation.

Finally, the last section of this chapter applies the performance evaluation framework to the reliability measure, discussing the possible uses, corresponding penalty curves and generalising functions, and illustrating with some examples.

6.2. REVIEW OF RELIABILITY CONCEPTS AND MEASURES IN WATER SUPPLY AND DISTRIBUTION

6.2.1. Introduction

Over the last few years, various techniques have been developed for the specific problem of quantifying the reliability of a water distribution system. One of the main difficulties with the concept of reliability is that there are many ways of expressing it and at least as many possible measures. An accepted certainty in the literature is the non-existence, as yet, of a single, universally established definition and measure of water network reliability.

To ensure a reliable delivery of finished water to the user, the distribution system is conventionally designed to accommodate a variety of expected loading conditions. For instance, water system components will be designed to satisfy daily maximum hour consumptions as well as daily or hourly averages and perhaps maximum instantaneous demands, including fire fighting flows which may be superimposed on those. Additionally, some provision may be made to accommodate *abnormal* conditions such as broken pipes and fittings, mechanical failure of valves or pumps, contamination accidents, power failures and unexpected demand levels or patterns [1].

Failure of the system will take place if any or several of those conditions occur in greater severity than allowed for by the designer. Most of the conditions listed above are normally classified as *mechanical failure*, while the very last one mentioned is the simplest form of *hydraulic failure*. In fact, most mechanical failures will result in hydraulic failure, but the distinction is made here since it is very often found in the literature.

[1] Failures due to water shortages at source are not considered in this study, which is chiefly concerned with the performance of the distribution network itself.

Reliability of a water distribution network is conventionally associated with the probability that a system performs its mission within specified limits for a given period of time in a specified environment (Mays, 1986). Walski (1987) attempts to be more pragmatic by pointing out that, however the reliability of a water distribution system is defined, it should be quantified through a measure reflecting the way in which water users are affected by it, such as the number of users with restricted or no service and the respective length of time the condition occurs. Xu (1990) has emphasised the importance of not only analysing the probabilistic nature of the occurrence of failures and their respective duration, but also the consequence of those failures, that is, the magnitude of supply shortfall related to different consumers distributed over the system.

This section will attempt to provide an overview of the most relevant concepts, beginning with those that seek a direct quantification of reliability properties, and subsequently moving on to some techniques that provide an indirect assessment of reliability by studying derived or *surrogate* properties of the networks.

6.2.2. Direct reliability measures

It has been mentioned before that the breakdown of a water supply and distribution system can be caused by mechanical as well as hydraulic failure. Mechanical reliability may be measured by the probability that the component or system being considered is operational at any time. The mechanical reliability of a network depends on the mechanical reliability of each individual component, and the way in which they are arranged together to make up the system. Hydraulic reliability, in turn, is a measure of the probability that each demand is adequately supplied at the required pressure. As mentioned above, it is clear that the former will have an effect on the latter, but of more importance to the hydraulic reliability in itself is the hydraulic performance of the network.

A variety of methods for calculating mechanical reliability is available, mostly based on topological considerations such as the expectation of the level of connectivity of the network for each specific configuration. Tung (1985) defined network reliability R as the probability that all demand points can be reached and, conversely, *unreliability* R' as the probability that any demand point cannot be reached. Wagner *et al.* (1988a), defined R similarly as the probability that all demand nodes are connected to a source, but introduced a nodal reliability R_n defined as the probability that the particular node is connected to a source.

It should be kept in mind that these types of measures reflect an analytical point of view which, although undoubtedly useful, leave aside engineering considerations of major relevance to the reliability of a water distribution system, such as the hydraulic characteristics of the links, the amount of unused or spare hydraulic capacity, the existence or placement of service reservoirs within the network, etc..

Mechanical reliability on its own can be used for assessing global system reliability by means of network connectivity considerations and the expectation of unreliable components. Exact calculation of reliability for a generalised network based on connectivity alone is a difficult problem, as Provan and Ball (1983), among others, have shown (Agrawal and Barlow, 1984).

Having reviewed some of the methods presented by Billington and Allan (1983) for the evaluation of reliability in electrical power systems, Tung (1985) discussed the suitability of the following techniques for calculating mechanical reliability of water distribution systems: a conditional probability approach, the cut-set method, tie set analysis, the connection matrix method, the event tree technique and fault-tree analysis. Tung selected the cut-set approach with first order approximation as the most computationally efficient method. Appendix B contains an introduction to the concepts and terminology of graph theory. Very briefly, the cut-set method is based on sets of components whose collapse cause the system to fail. A cut-set is a set of edges whose removal from a connected graph leaves it disconnected. A cut-set can be identified between any two mutually exclusive subsets of nodes by means of the minimum number of edges whose removal disables all paths between those two subsets.

Minimal cut-sets are defined as cut-sets that only cause system failure if *all* the set's components fail. The probability of failure of a given minimal cut-set C_i (from the universe of M possible minimal cut-sets) can be calculated from the unreliability R'_n of the nth component thus:

$$p(C_i) = \prod_n R'_n \tag{6.1}$$

The failure of any minimal cut set causes the system to fail, and so does any combination of minimal cut sets belonging to M. The system unreliability can therefore be reached through:

$$R' = p\left[\bigcup C_i\right] \tag{6.2}$$

The evaluation of the above probability may be found in Billington and Allan (1983), for example. Like most measures based on topological considerations alone, this method overlooks hydraulic capacities of the links and implicitly assumes that it is possible to satisfy demands so long as at least one edge in the cut-set is operative, therefore assuming that the hydraulic balance will be attainable in those conditions. As Jacobs and Goulter (1988) point out, reliability of topological connection to a source is only a necessary condition for a reliable water supply and does not reflect the true reliability issue of the water supply and distribution system.

Quimpo and Shamsi (1989) presented another application of minimal path-sets and cut-sets, regarding the distribution network as a directed stochastic network. Reliability is calculated at various demand points and a reliability surface is used to highlight the areas in greater need of improvement. One important limitation of such approaches is the need for exhaustive enumeration of system paths or cut-sets, a very time-consuming task at best. For that reason it is unrealistic to consider their use in reliability evaluation of all but the simplest of networks, their applicability in *real life* cases being greatly restricted.

In all the above methods, reliability is evaluated by means of some discretisation of the network into its components or sets of them. Some mention should be made, however, of a category of methods that assess the reliability of the system as a whole, also known as lumped methods. One of the earliest references is a technique developed by Damelin *et al.* (1972) to evaluate the reliability of a supply system subject to random failure of pumps for a given level of consumer demand. The consequence of the pump failure is estimated through Monte-Carlo simulation, and the indicators chosen to characterise reliability were the amount of shortfall and the frequency of such shortfall.

Tangena and Koster (1984), Shamir and Howard (1985) and Mays and Cullinane (1986) have all emphasised the advantages of a system availability approach to assess bulk supply reliability. The former presented a method in which each component of the system is assumed to be in one of two states: a working state and a failure state, which are characterised respectively by mean time to failure and mean time to repair. The system availability is calculated through a fault tree analysis, and the reliability is associated with both the probability of complete supply and the average quantity not supplied.

Germanopoulos *et al.* (1986) evaluated reliability of supply and level of service in water supply systems by means of a contingency analysis based on network simulation and probability analysis. System reliability is measured by the frequency and duration of critical failure of the system both during mains bursts and contamination events at source. Frequency-duration analysis is performed with the probability of the failure events described by a Poisson distribution and the duration of the failure events modelled using the exponential distribution. Network simulation is deployed to assess the consequence of the failure event, with pressure-dependent demands being used to more realistically simulate the behaviour of the reduced systems (Germanopoulos, 1988).

In Hobbs and Beim (1988a), the unavailability and expected unserved demand of a complete water supply system with random demand, finished water storage and unreliable storage components are assessed, with the objective of promoting better understanding for operational rules as well as the optimisation of capital expenditure in expansion strategies. Those reliabilities are estimated through the use of modified frequency-duration analysis, by calculating how often demand exceeds available capacity and then comparing water storage with how long the deficits last. Hobbs and Beim (1988b) utilise a Markov chain approach to verify the previous models. A reliability model which represents demand changes, failure and repair of capacity components and stream flows as independent Markov chains is presented.

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Hydraulic reliability, translated basically as the likelihood that each node of the network will provide the required supply at sufficient pressure, is not easily calculated. Many difficulties arise in the adequate calculation of probabilities, not least due to the fact that the performance of a looped network with a failed component is ultimately impossible to infer with exactitude from the normal mode of operation, given the re-distribution of flows that occurs in those circumstances. For each possible or probable failure event, the reduced network must be analysed for hydraulic adequacy. This is further complicated by the fact that the analysis of the hydraulic equilibrium of a water distribution network, with its system of non-linear equations, is less than easy to manipulate and normally demanding in computational terms. The size of the problem escalates quite easily: if the network is considered to be subject to random failure of components, it is necessary to analyse an exhaustive number of reduced configurations if the exact global probability of failure for the network is to be reached. Given the large number of possible configurations (up to 2^N for a system with N links) and the heavy computational load incurred by network analysis, systematic calculations are normally unfeasible for any realistic network.

The ensuing need for some kind of approximated assessment of the above mentioned probabilities gave place to some methods documented in the literature. Wagner *et al.* (1988a) have defined reliability as the probability that all specified nodal demands can be supplied. Equally, Bao and Mays (1990) have defined reliability as the probability that the system can provide the necessary flow rate at the required pressure. Following Carey and Hendrickson (1984), Fujiwara and Silva (1990) have defined reliability in terms of the expected minimum total shortfall to the total demand. The *unreliability* of the network is taken as the ratio between the expected minimum total shortfall and the total demand. The reliability is naturally defined as the complement of the unreliability.

Wagner *et al.*(1988a) developed a method to calculate the reliability of distribution system by applying certain analogies with communications systems as discussed, for example, in Agrawal and Barlow (1984), making use of the concepts of reachability and connectivity. *Reachability* of a demand node is defined as the property of being connected to at least one source, while *connectivity* of a network measures the property of every one of its nodes being *reachable* (or connected to at least one source). The probability of connectivity between the demand nodes and the source nodes is computed by network reduction, and hydraulic adequacy is tested using a capacitated network.

To obtain the probability of sufficient supply, Wagner *et al.*(1988a) assign a given capacity to each link, based on an assumed maximum hydraulic gradient of 0.01. It is noted that pipes in a distribution network do not actually have a capacity as such, since it is the available pressure that will condition the flow. They then determined whether each reduced network configuration can provide enough flow by modelling it as a maximum flow network problem. Determining maximum flow solutions of a network subject to maximum link capacities is a well documented problem with efficient LP solutions such as given by Bazaraa and Jarvis, 1977. Knowing the probability that the network will be in each reduced configuration, it is then theoretically possible to calculate the probability that the network will satisfy the demands. This probability is given by the complement of the joint probability of the configurations that cannot supply the required flow. Therefore, the reliability is the complement of the probability that the network will be in any of the reduced configurations which cannot supply the required flow.

Bao and Mays (1990) give a measure of reliability based on the probability that the system is able to provide the demanded flows at the required pressure heads. Although this definition is not far from the aforementioned probability of sufficient supply, the approach followed here was totally different. The reliability estimation of a water distribution system is thought to be subject to uncertainty as the random nature of water demands, required pressure heads and pipe roughnesses is recognised. Instead of quantifying the hydraulic failure resulting from mechanical failure, the authors measured hydraulic reliability by means of probability distributions assigned to the nodal demands and their respective pressure requirements.

The proposed method is based on Monte-Carlo simulation, generating the random numbers that drive a demand-based hydraulic network simulator. Nodal demands are made to be met regardless of the actual pressure values, and the hydraulic reliability is defined as the joint probability that the actual nodal pressure head satisfies the pressure requirement at the node. The nodal hydraulic reliability R_n at the generic node n is thus expressed as a function of the pressure head requirement H^{req}:

$$R_n = P(H_n > H_n^{req}) = \int_{H_n^{req}}^{\infty} F_n(H_n)dH_n \tag{6.3}$$

$F_n(H_n)$ is the probability density function of the supplied pressure head. Bao and Mays suggest some distributions but not unexpectedly warn of the inherent difficulties of estimating parameters whatever the choice.

In order to calculate a system-wide reliability value, and given the difficulty in establishing the dependencies between the nodal reliability values, the authors propose some heuristic methods. These take for the system value either the minimum nodal reliability (i.e., system reliability is as good as its least reliable node), the arithmetic mean of all nodal reliabilities, or finally a weighted mean of the same values using the nodal consumptions as weights.

Fujiwara and Silva's approach, mentioned earlier, employs a method for reducing the number of failed configurations to be analysed. It is assumed that simultaneous link failures are unlikely and therefore negligible, and the analysis only considers system configurations with one link failed. For each configuration, the maximum flow delivered is approximated using a maximum flow network model with capacities assigned to each link. How these capacities are decided is discussed later when the optimisation of network reliability is analysed. Knowing the maximum flow delivered, the minimum shortfall for each state is given by the difference between the total demand and the maximum flow delivered. The expected minimum total shortfall for the system is then given by the sum of the shortfall for each state, weighted according to the respective state probabilities. Reliability is, as mentioned before, given by the complement of the unreliability, defined by the ratio between the expected minimum total shortfall $E[S]$ and the total demand D :

$$R = 1 - \frac{E[S]}{D} \tag{6.4}$$

6.2.3. Reliability of optimal layouts

Before proceeding to analyse the approaches to reliability that are based on surrogate measures, it is important to introduce at this point some aspects that are very relevant to the discussion. The layout of a water distribution system is one of the most influential factors to be taken into account when its reliability is assessed. Goulter (1992) argues that the fundamental shape of the layout actually establishes an upper bound for the system reliability.

Most of the approaches published for reliability calculation and particularly its maximisation are based on fixed layouts, not least because the problem of optimising the layout of a water network is rather complex. Some proposals are made in the literature for this purpose, invariably involving a fixed number of nodes in pre-specified positions, with the existence of links between them being one of the variables at stake. The introduction of extra nodes seems not to have been explored so far.

One of the most important factors in this type of discussion is of course that the optimal design of water distribution networks has traditionally been driven primarily by cost considerations (e.g., Alperovits and Shamir, 1977, or Quindry *et al.*, 1979, 1981). However, as pointed out by Templeman (1982), the minimum cost objective of the optimisation solution will by its very nature remove any possible redundancy by eliminating any unnecessary components to a direct-path supply. It is a well-known result that the least cost optimisation of a looped network will yield a tree-shaped configuration, a single large pipe being normally cheaper than a combination of smaller diameters for supplying the same flow rate. Although measures for preserving the looped structure can be taken, such as imposing a minimum pipe size, the resultant network will probably be no more than an implicit tree, and its reliability of supply will hardly be suitable. It is in this framework that the optimisation of layout to introduce the desirable properties — namely, reliability — appears as an important if complex field of study.

Layout optimisation models are usually categorised in at least two main camps. The first category concerns those models that begin by considering all the potential links of the network and then proceed to delete some of these links in order to reduce costs while satisfying given reliability criteria. Morgan and Goulter (1985) and Awumah *et al.* (1989) present two of the proposals in this category.

Morgan and Goulter's approach is an iterative procedure in which a relatively large number of combinations of load patterns and link failures can be considered simultaneously with the nodal pressures, the resulting designs being checked by a network simulation model. The method sizes the network through a linear programming formulation of the type proposed by Alperovits and Shamir (1977) based on the lengths of pipe in each link that are to be replaced by either the next size up or the next size down in the following iteration [2]. This keeps the number of variables constant throughout the analysis, which facilitates the examination of several demand situations for each tested configuration. This method purports to achieve the incorporation of reliability precisely by testing a variety of demand situations (including exceptional demands) and potential link failures for each design, a process which gets excessively heavy for larger networks. Unfortunately, the method of iterating between network configurations relies on a purely heuristic choice of which link(s) to delete next, which the designer must make based on weights attributed to the links. The process is sequential and is likely to bypass the global optimum.

Awumah *et al.* (1989) devised a method which assumed the initial nodal heads and solved a 0-1 integer programming problem, with the reliability-related requirements that each node must be connected by at least two pipes, and that no node is cut off by the failure of a single link. The method needs several candidate diameters for each link to be specified and therefore generates a large number of variables.

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2 Since it is either one or the other, one of the two will be zero each time round and the number of non-zero variables equals the number of links.

A second category of models for layout optimisation utilise the opposite approach by starting off with a simple spanning tree and then proceeding to add *redundant* links in order to meet certain reliability criteria, while keeping the cost of the network within a specified boundary. Models belonging in this category are proposed by Rowell and Barnes (1982) and Loganathan *et al.* (1990).

Rowell and Barnes (1982) have developed an approach which partially overcomes the limitation of homing in on local rather than global reliability. The method consists of two stages. In stage 1, the network is solved to obtain a minimum cost solution, defining a tree shaped network in the process. In stage 2, the tree network is augmented with pipes selected optimally [3] from the non-tree links to provide an alternative supply path to each demand node. This is a rather difficult problem to solve. Rowell and Barnes used a 0-1 integer programming formulation which also determines the diameter of the added redundant links.

This method's main weakness is the flow adjustment process in the second stage, since the formulation of the non-linear minimum cost flow model is based on the assumption that all the links in the initial tree will have the same hydraulic gradient. As remarked by Goulter and Morgan (1984) or Kessler *et al.* (1990), this assumption is incorrect and does not guarantee hydraulic consistency, in the absence of any mechanism to ensure that any loop and path constraints are not violated following the addition of redundant links. A second major disadvantage is the use of integer programming to solve the second stage, which is not efficient for larger networks.

Loganathan *et al.* (1990) utilised a similar approach, but guaranteed the hydraulic consistency of the network by making only minimal adjustment to the initial near optimal core tree design, when the network is redesigned following the addition of redundant links. Another important

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3 Each pipe of the tree network is sequentially removed and the corresponding set of isolated nodes is identified. For each of those sets, a minimal set of pipes necessary to reconnect the network is determined, and the pipes sized to meet the demand of the set of isolated nodes.

difference from Rowell and Barnes's technique is the fact that the pipe sizes are not determined simultaneously with the flows neither in the design of the core tree network nor in the final looped network.

Here, the flows are first calculated and then the pipe sizes are found using the linear programming step of the linear projected gradient method. In order to design the spanning tree, and if it is the case of a single source network, the initial tree-type solution is based on a minimum spanning tree (such as suggested by Templeman, 1982a, for example), with the flows determined from the continuity equations. For multiple source networks, a linear minimum cost flow model is used to determine the flows and directions in each link and to obtain an initial spanning tree.

Having chosen the initial tree in either case, the pipe sizes are then calculated by linear programming. Each potential link not already in the current tree is then sequentially added to it, while simultaneously removing a link in the resulting loop with the objective of obtaining a less expensive spanning tree design. When no further such substitutions are possible, the process yields what is a near optimal core tree.

Tanyimboh (1993) points out another major distinction between Rowell and Barnes (1982) and Loganathan *et al.* (1990): the fact that the latter formulated the problem of finding redundant links based on connectivity alone. The resulting 0-1 integer programming problem is thus solvable by an LP-based heuristic, as can be found in detail in the original paper and its references. The ensuing looped network is then redesigned by LP while keeping changes to the core tree to a minimum, a process which needs some judgement by the designer.

The method appears to find good solutions, as demonstrated by a set of examples in the original publication. Complex problems seem to be successfully decomposed into efficiently solvable sub-problems. In particular, it may be noted that the problem of finding redundant links is not solved with an integer programming algorithm. However, the cost of constructing the looped network is kept down by the use of mostly minimum size pipes for the redundant links. Such use of minimum diameter pipes is questionable, as pointed out by Wagner *et al.* (1988a), in the sense that the new links thus sized may not be introducing much additional redundancy, and will be doing so in an unquantified manner.

6.2.4. Indirect reliability measures

The explicit definition of reliability in water distribution systems, especially those that include loops in their configuration, is definitely a difficult task. More so is its quantification for most purposes, including that of the present work. Not unexpectedly, some authors have followed a different path by trying to evaluate reliability indirectly using parallel approaches, which to a certain extent bypass some of the problems highlighted earlier. Indeed, as seen in the previous section, connectivity concepts are used in a surrogate manner to provide the basis for some of the optimal reliability layout methods described, particularly that of Loganathan *et al.*.

Water distribution networks with high levels of cross-connectivity between source and demand nodes, and among the demand nodes themselves, have a high potential to maintain service to all nodes should a particular link within the system fail (Goulter, 1992). This ability is represented by the potential for as many alternate paths from the sources to the sinks as possible, notwithstanding the necessity to provide sufficient hydraulic capacity in those alternate paths. However, assuming that each alternate path does have the necessary capacity to supply the flow, the greater the number of alternate pathways in the network, the greater is the ability of the network to perform adequately — by meeting the volumetric water demand at adequate pressures in the occurrence of failures within the network.

Engineering practice has addressed this requirement in the design of urban water distribution systems by the inclusion of looped sets of links in the networks. The loops within the networks theoretically provide alternative pathways from the sources to the sinks through the network, should a particular path be unavailable due to the failure of a pump, pipe or other component in that path. In order to make those alternative paths sufficient in capacity to replace the one that has failed in terms of adequately satisfying the same demands, the system is usually simulated by the designer under a range of loading conditions and component failure, which cannot be exhaustive in itself but is deemed to represent, to the designer's experience or knowledge, the most likely and/or the most demanding conditions faced by the network.

The existence of these backup or alternate supply pathways constitutes redundancy in the distribution system. Reliability, in its most general sense, defines how well the system performs in meeting the demands upon it. It can therefore be seen that redundancy, as it represents the presence of alternate supply paths through the network, and the associated ability to maintain adequate service with some components out of service, is a major contributor to reliability. As Goulter *et al.* (1992) point out, it is this almost intuitive relationship between reliability and redundancy, and the nature of redundancy in water distribution networks, that led to some form of redundancy measure being pursued as a surrogate for reliability. This is an important notion that will be retained as the basis for the subsequent work on reliability developed in this chapter.

The indirect measures of reliability proposed in the literature are mostly based on redundancy considerations and have been either derived from graph theory concepts or from the principles of entropy maximisation developed in information theory and thermodynamics. A review of the application of the former to reliability analysis in water supply follows, while the latter will be treated separately in subsequent sections, given its importance in the context of the present study.

One of the earliest applications of graphs is described by Elms (1983), who makes a general attempt to quantify the degree of connectivity within existing networks, both among the nodes themselves and among groups of nodes, by use of a clustering algorithm.

Goulter (1988) adapted the methodology to the more specific case of water distribution networks, by modification of the connectivity terms to ensure that loops occurred in the system. Goulter's measure was able to quantify the level of connectivity in a network and therefore differentiate between two networks in terms of their connectivities, which amounted to comparing their respective degree of redundancy. However, the method is computationally impractical and incapable of enumerating the number of paths between any two nodes (one of which would be a source). Furthermore, it had no way of incorporating the hydraulic capacities of the links, perhaps the greatest drawback of some of the indirect measures using clustering approaches.

Subsequently, in a general graph-theory based study of how network layouts might be decomposed for reliability analysis, Jacobs and Goulter (1988) determined the layout characteristics of an optimally reliable network. An optimally reliable network in this context is the network for which, all other conditions being equal, the reliabilities in terms of connectivity could be maximised. This network was found to be a regular graph, that is, a layout in which the same number of links are incident on each node, based on the application of the graph theory result that shows damage-resistant optimal graphs to be regular in degree at all nodes [4]. In other words, for a given number of links in a network, maximum reliability from this point of view is achieved when each node has an equal number of links incident upon it. As the number of links in the network increases, the connectivity, and therefore the redundancy of the network, also increases. However, the maximum reliability will still always occur when regularity is maintained or, if the number of links in the network is not enough for exact regularity, the network is kept as close as possible to being regular.

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4 See appendix B for graph-theory terminology.

Jacobs and Goulter (1989) extended this work to optimising redundancy in water distribution system designs and examined how the shape of the network changed as the target of how many links should be incident on each node was varied. A model where the layout is the only concern is presented. A regular network is generated using integer goal programming, minimising the differences between the degrees of the nodes of the network. The resulting network is then examined for weaknesses, such as parts of the network becoming detached from the rest of the system due to unwitting elimination of all its connections to it. Constraints relating to the network can then be introduced at this stage and the model is re-run as many times as necessary to eliminate all weaknesses. This model does not take into consideration any hydraulic constraints, pipe lengths, sizes and capacities, or costs. A heuristic method for weighting each node by the inverse of its demand is utilised, to less than conclusive results, particularly given the above mentioned weakness of not explicitly considering the need for adequate hydraulic capacity in the links.

Kessler *et al.*(1990) introduce a different approach, where two trees are simultaneously used to design a network that is invulnerable to a single failure. In the first or layout stage, two spanning trees are generated for the network, in such a way that they overlap and jointly guarantee the existence of an alternative path to each demand node, in the event of a single link or node failure. The trees are generated using appropriate graph theory algorithms.

In the second stage, the minimum hydraulic capacity of each path is determined using a linear programming model to calculate the pipe sizes. A measure of invulnerability is given to the network by designing each tree so that it can supply all demand flows on its own and at adequate pressure. In a third stage, the solution is tested by a network solver for various demand patterns.

This approach has the advantage of approaching jointly the issues of layout-related reliability and component-related reliability, even though in a sequential manner. Since both paths to the demand nodes are actually hydraulically designed, it is possible to control the extent to which each of the paths to each node can be trusted. The method is however limited to single-source networks and there is no means of determining the best pair of trees prior to a full design, which inevitably makes it time-consuming if exhaustively employed.

Tanyimboh (1993) points out a slight theoretical weakness in the formulation. There are no loop equations in the constraint set, which consists of length constraints and a lower bound on the head at each node of each tree. In fact, there is no guarantee of conservation of energy around the loops of the network, as the nodal pressure constraints are inequalities. The example is given of any node with two non-overlapping supply paths, which forcibly start from the same source. It becomes obvious that the head loss around the loop defined by these paths will be zero only if the allowable head loss is the same for both paths, and the nodal pressure constraint of each tree is active or the two slacks are the same at the solution. There is however no guarantee that these conditions will be met in general.

None of the two methods present a direct quantification of the invulnerability that they are trying to achieve, and neither therefore lends itself to direct optimisation on that basis. On the same principle, none of the indirect reliability approaches reviewed so far provides a quantifiable measure of reliability, or at least redundancy, in such a way that it can be used as a standardised means of assessing any given network, existing or to be designed. The exception would be the aforementioned method by Goulter (1988), which is unfortunately thought to be computationally impractical.

The other main technique for indirect assessment of reliability in water distribution is relatively recent and involves, as mentioned previously, the application of entropy maximisation concepts derived from information theory and thermodynamics. The relevance of this type of method to the present study justifies a special treatment, as it appears to offer sufficient potential for the development of a reliability performance measure . The next sections are therefore devoted exclusively to that subject, beginning by a review of the published works in the field.

6.3. ENTROPY AS AN INDIRECT MEASURE OF NETWORK RELIABILITY

6.3.1. Introduction

The previous section reviewed some of the most important methods for reliability evaluation mentioned in the literature. The review has divided the available techniques into direct and indirect methods, and discusses how the concepts of reliability and redundancy can be associated and how both may be better evaluated using indirect techniques. The use of maximum entropy flows is one of the main methods for indirect or surrogate evaluation of reliability. It has been deliberately left out of the previous review so that it may be analysed in greater detail in this section, as it provides the basis for the reliability performance evaluation proposed in this work.

It has already been remarked how the concept of redundancy is related to the reliability of the networks, and how it may provide a suitable basis for the development of a quantifiable measure. The main requirements for such a measure would be that it should be able to quantify redundancy in such a standardised way that comparisons between different networks become possible, and that redundancy is the same for two networks with the same layouts and different but directly proportional pipe sizes. The redundancy should also reflect the diversity of paths between supply and demand, and be equal to zero in case there is only one path. Equally, the redundancy at a particular point of a network, such as a demand node, should increase with the growth in number of incident links.

The present section begins by briefly introducing the most relevant points of the entropy maximisation methodology. The evolution of entropy maximisation applications to reliability assessment in water distribution networks is traced and the methodologies so far available in the literature commented upon. A new formulation is then proposed which corrects or completes some of the published methods. This is followed by a discussion of the suitability of entropy maximisation for reliability evaluation, which raises some points of relevance for the subsequent use of the methodology as a performance measure.

6.3.2. Entropy maximisation for network flows

Entropy is a concept generally utilised in thermodynamics to describe the state of disorder, randomness or lack of information about the microscopic configuration of a system. The application of the same concept to information theory by Shannon (1948) and further work by Jaynes (1957) yielded the theoretical framework that provides what is "(...) essentially a general technique for guarding against bias" (Tribus, 1961).

Shannon's entropy is formulated in order to enable the analyst to make the best, unprejudiced (or unbiased) estimate in the presence of uncertainty, without introducing unconscious arbitrary assumptions. It is a quantitative measure of the uncertainty (or, conversely, amount of information) in a probability distribution, therefore allowing for quantified comparison between the uncertainties associated with different distributions.

Shannon proposed the following quantity, S, to measure the entropy of a given probability distribution:

$$S = -K\sum_{i=1}^{N} p_i \ln p_i \qquad (6.5)$$

where S is the entropy, K is an arbitrary positive constant and $p_i, i = 1,\ldots,N$ is *a finite probability scheme*, that is, a set of events, and the probabilities they are associated with, that are exhaustive and mutually exclusive. In other words, a system such that the outcome of any trial must always be one of the events in the set, and only one. Two basic properties of such a scheme are that all probabilities are non-negative and satisfy normality, adding up to unity.

Tanyimboh (1993), in a rigorous presentation of the entropy formulation and its application to the optimisation of reliability in water distribution networks, lists the following relevant properties of the entropy function [5]:

- The entropy of a system is greater or equal to zero, $S \geq 0$, with the latter occurring when there is only one possible outcome (N=1, p_1=1) and therefore no uncertainty.

- The entropy function is continuous with respect to all its elements for any N and independent of the order in which the probabilities p_1 enter Eq.6.5. The entropy function is also a concave function.

- $S(p_1, p_2, \ldots, p_n) = S(p_1, p_2, \ldots, p_n, 0)$ or the value of S is not altered by adding impossible events (p=0) to the scheme being considered.

- The maximum value that S can assume $Max(S) = S(U)$ corresponds to the entropy of a uniform distribution where $p_i = 1/N, \forall i \in N$, which indeed maximises uncertainty or lack of information about its events.

- The joint entropy of a scheme composed of two finite probability schemes O_1O_2, or a *compound probability scheme*, is given by:

  $$S(O_1O_2) = S(O_1) + S(O_2 | O_1) \tag{6.6}$$

  and is invariant with respect to the relative positions of the two schemes in the formulation. If O_1O_2 are mutually independent, then $S(O_1O_2) = S(O_1) + S(O_2)$.

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[5] The properties are presented here in result form only. The proofs may be found in Tanyimboh (1993) or his references.

A uniqueness theorem (Khinchin, 1953) shows that Shannon's entropy function for a finite scheme is the only possible function defined for any integer N and for all values of a finite probability scheme p_i, to have the above properties, which Tanyimboh (1993) shows to be essential in view of the actual meaning of the entropy concept in the desired context.

Shannon's entropy may be used to measure the uncertainty of a given probability distribution provided this distribution is known beforehand. Jaynes (1957) showed that Shannon's entropy can furthermore be used for logical inference, stating the *maximum entropy formalism*: in making inference on the basis of partial information, the probability distribution that has maximum entropy must be used, subject to whatever is known. This is the only unbiased assignment possible, any other assignment entailing the arbitrary assumption of information not known.

The significance of the maximum entropy formalism in the context of inferring network flows and maximising their redundancy/ reliability is shown by Tanyimboh (1993). If such a problem can be formulated in probability-like terms, and if the maximum entropy formalism makes it possible to find the most unbiased probability distribution for a system, it seems logical that a maximum entropy distribution provides the only solution where any state that is not excluded by the available information is ascribed a non-zero probability, and conversely, non-zero probabilities are only assigned to those states. Tanyimboh argues that it appears safe, in order to achieve reliable designs, to size the pipes to carry flows that are maximally noncommittal to factors that cannot easily be predicted, subject as much as possible to whatever information is available.

6.3.3. Review of entropy applications in water supply and distribution

The methods based on the application to water distribution systems of entropy maximisation concepts derived from information theory have been introduced by Awumah, Bhatt and

Goulter (1990) for the evaluation of the degree of redundancy present in a water distribution system, and further improved by the same authors (1991, 1992, 1994). Awumah and Goulter (1992) have approached the same subject with a view to formulating an optimum layout/ reliability problem. Tanyimboh and Templeman (1993a, 1993b) and Tanyimboh (1993) later presented a more structured and refined analysis, with the objective of optimum design, also resolving some logical flaws in the initial approach of Awumah et al.

The basic idea behind this approach was to obtain the distribution of flows in a network which guarantees the greatest uniformity between all the supply paths to all the nodes, therefore minimising the expected shortfall in case of a pipe breakdown. This measure reflects both the topology of the network and the magnitude of the flows in the links, while leaving aside pressure-related considerations and the effects of pipe lengths, hydraulic gradients, etc.. In essence, what is pursued with this approach is a degree of redundancy in the way each demand point is supplied, based on the already introduced notion that redundancy increases reliability.

This "most uniform" flow distribution would correspond to the highest possible value of an entropy function reflecting, or measuring, the uncertainty generated by the different paths of supply to each and every node of a network. The first step in this process therefore consists of identifying the entities – the probability spaces – over which the entropy would be calculated, and then formulating an appropriate entropy function that lends itself to maximisation within the principles of Shannon's theory.

Awumah *et al.*(1991) proposed the following function for measuring the redundancy of a water distribution system:

$$S = -\sum_{ij \in IJ} \frac{q_{ij}}{Q_0} \ln \frac{q_{ij}}{Q_0} \qquad (6.7)$$

S represents the entropy of the network, which is considered to measure its redundancy. IJ is the set of all the links in the network, q_{ij} is the flow in link ij, and Q_0 is the sum of the link flows as follows:

$$Q_0 = \sum_{ij \in IJ} q_{ij} \tag{6.8}$$

The above definition of entropy is meant as an application of Shannon's formula. As seen before, though, Shannon's entropy is defined only for mutually exclusive probabilities or events. Awumah and Goulter chose the quantities q_{ij}/Q_0, which, despite being probability-like, are not necessarily mutually exclusive, as indeed Goulter (1992) points out. In a network with links connected in series (Fig.6.1), the flows in any pair of links in the series are not independent. In fact, the flow in any link of the series (except for the one at the upstream end) is dependent at least partially on flows in upstream links. If these flows are not independent then they are not mutually exclusive, which invalidates Eq.6.7 in terms of the requirements of Shannon's entropy.

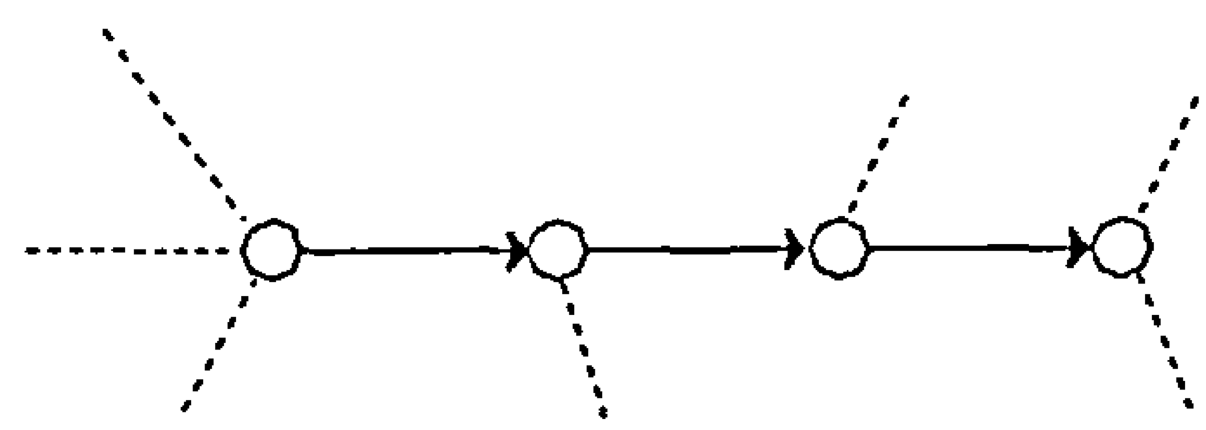


Fig.6.1 - Series connected links

Awumah *et al.*(1991) try to get round this difficulty by obtaining a node-based expression, with the help of the following substitution applied to Eq.6.7:

$$\frac{q_{jn}}{Q_0} = \frac{q_{jn}}{Q_n}\frac{Q_n}{Q_0} \quad \forall n \tag{6.9}$$

Q_n being the sum of the link flows entering node n. The transformed but equivalent equation is therefore:

$$S = \sum_{n=1}^{NN} \frac{Q_n}{Q_0} S_n - \sum_{n=1}^{NN} \frac{Q_n}{Q_0} \ln \frac{Q_n}{Q_0} \quad \forall n \tag{6.10}$$

where

$$S_n = -\sum_{j \in U^n} \frac{q_{jn}}{Q_n} \ln \frac{q_{jn}}{Q_n} \quad \forall n \tag{6.11}$$

is the entropy or redundancy of node n and U^n is the set of upstream nodes of link inflows at node n. It must be noted that the substitution does not solve the initial flaw of non-normality of the supposedly probability-like flow ratios Q_n/Q_0. This expression not accounting for the interactions between adjacent nodes in a network, Awumah *et al.*(1991) go on to propose the following correction:

$$S'_n = S_n + \sum_{j \in U^n} t_{jn} S'_j \quad \forall n \tag{6.12}$$

in which t_{jn}, varying between zero and one for all n, is the fraction of the modified entropy S_j' of node j belonging to U^n that is passed on to node n and is:

$$t_{jn} = \frac{q_{jn}}{Q_j} \quad \forall n, \forall j \in U^n \tag{6.13}$$

In order to calculate the modified entropy S'_n for any node, its preceding nodes must have been calculated beforehand. If S_n is replaced by S'_n in Eq.6.10, the following expression for the network entropy is finally reached:

$$S = \sum_{n=1}^{N} \frac{Q_n}{Q_0} S'_n - \sum_{n=1}^{N} \frac{Q_n}{Q_0} \ln \frac{Q_n}{Q_0} \quad \forall n \tag{6.14}$$

This expression using S'_n was found by Awumah *et al.* (1991) to yield higher network entropy values than when using S_n. However, the shortcomings of Eq.6.7 are never really addressed in any of its derivatives. Furthermore, none of the expressions take into consideration the uncertainty due to the external supply or demand flows at the nodes. As seen subsequently with Tanyimboh (1993)'s more rigorous approach, proper understanding of the flow-splitting or flow-joining processes that generate the uncertainty this approach is attempting to measure does not seem to be translated by Awumah's formulation.

Further sophistications are proposed by Awumah *et al* (1991), Awumah and Goulter (1992), to account for the interdependencies between different paths supplying the same node, which may have some links in common, and to take into consideration the possibility of flow reversal. These adjustments, however, just as all the other corrections proposed by Awumah *et al.* for their formulation, stem basically from the inadequacy of the original probability spaces over which it operates, and would appear to be the result of semi-empirical considerations. Tanyimboh (1993) clearly demonstrates the inadequacies of this formulation.

The work of Tanyimboh and Templeman (1992, 1993) and Tanyimboh (1993) introduced a more systematic approach to this field. Having established the correct entropy formalism framework, Tanyimboh (1993) shows how the concept of entropy for finite probability schemes may be applied to general flow networks. The problem centres around the correct modelling of flow ratios as probabilistic quantities and reaching a suitable formulation of entropy for those probabilities. Using the relative frequency interpretation of probabilities, various ratios of flows in a general network are tested. The relative frequency of an event is the frequency of number of times the event occurs divided by the total number of occurrences of all events in the set. Unless there is careful choice of the sets of events, probabilities obtained in that way do not always represent a finite scheme. Indeed the shortcomings identified in Awumah *et al.*'s formulation have been seen to originate in this area.

Tanyimboh begins by analysing parallel networks, or networks where the links connect source nodes exclusively to demand nodes and vice-versa (Fig.6.2). For that reason, the sum of the link flows of a parallel network equals the total supply or demand. This type of

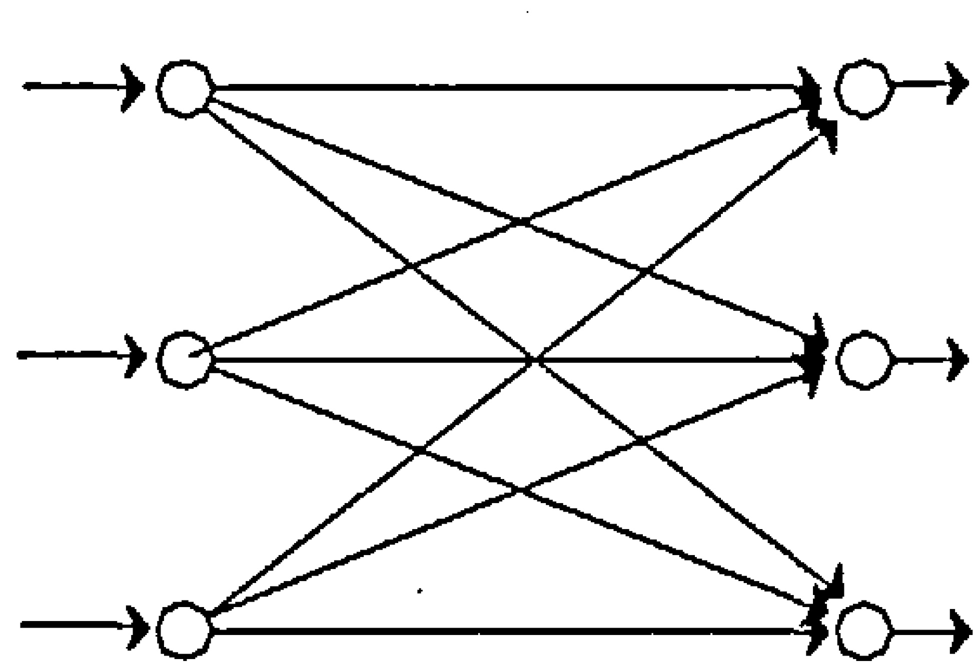


Fig.6.2 - Parallel network

network lends itself well to the entropy approach as several finite probability schemes can be clearly defined for such networks by normalising on the total sum either the supply flows, the demand flows or, more interestingly, the link flows. Well established results of entropy maximisation are therefore directly applicable to the resulting single space probability scheme, which provides a good starting point to the analysis.

A multiple-space probability model is also formulated for parallel networks by defining separate finite schemes for each node. The flows leaving the node, including eventual demands, are normalised on the basis of the total flow through the node. Conversely, the flows entering the node, including any external supplies, can be normalised with the same total nodal flow. The multiple-space formulation raises the question of conditional probabilities, since conditions at the general node of even purely parallel networks are not independent from either the demand node downstream or the supply node upstream of it, whichever the case.

Because the flows in series-connected links are not mutually exclusive, more general networks require precisely such a conditional probability model. Unlike parallel networks, the amount of flow passing through a given node is generally unknown, at least in the type of problems where the present technique is applicable. Equally, the probability of flow reaching a node will not be known. Tanyimboh proposes the following multiple-space probability model as the basis for a new formulation of the network entropy function. Albeit innovative in its systematic approach, it is worth noting that this formulation is still based on the node as the elementary level where the finite schemes for flow distribution are defined, and whose elementary entropy values are then weighted and averaged across the network for a global network value.

Let Q_n and Q_0 be the total flow through a node and the total flow through the network respectively:

$$Q_n = \sum_{j \in U^n} q_{jn} = \sum_{k \in D^n} q_{nk} \quad n = 1, \ldots, N \tag{6.15}$$

$$Q_0 = \sum_{n=1}^{N} q_{n0} = \sum_{n=1}^{N} q_{0n} \tag{6.16}$$

where q_{0n} is the supply at node n, q_{n0} the demand, q_{jn} is the flow in the link from node j to node n and q_{nk} the flow from n to k. U^n and D^n are the sets of upstream and downstream nodes of node n, including the fictitious super-nodes where supplies originate and demands flow to. The fraction of total supply provided by the supply at node n and the fraction of total demand consumed at n are given respectively by the next two expressions:

$$P_{0n} = \frac{q_{0n}}{Q_0} \quad \forall n \tag{6.17}$$

$$P_{n0} = \frac{q_{n0}}{Q_0} \quad \forall n \tag{6.18}$$

The following two finite schemes are proposed for each node of the network:

$$p_{jn} = \frac{q_{jn}}{Q_n} \quad \forall n, \forall j \in NU_n \tag{6.19}$$

$$p_{nk} = \frac{q_{nk}}{Q_n} \quad \forall n, \forall k \in ND_n \tag{6.20}$$

From the definition of U^n and D^n it follows that the above two schemes include respectively the supply and the demand at n, $p_{0n} = \frac{q_{0n}}{Q_n}$ and $p_{n0} = \frac{q_{n0}}{Q_n}$, $\forall n$.

The concept of entropy can finally be applied to the resulting multiple space probability distributions by means of the conditional entropy formula for compound probability schemes. Tanyimboh expresses the entropy of a network based either on nodal inflows or on nodal outflows, as the sum of elementary nodal entropies (plus a term for demands or source flows, respectively). Beginning with the inflow-based network entropy, S^i:

$$S^i = S_0^d + \sum_{n=1}^{N} S_n^i \qquad (6.21)$$

in which the first term is the entropy of the finite scheme defined for the demands in Eq.6.18:

$$S_0^d = -\sum_{n=1}^{N} P_{n0} \ln P_{n0} \qquad (6.22)$$

and the second term is the entropy of inflows including any supply, as defined in the finite scheme of Eq.6.19, conditional upon the probability of flow reaching the node, given by the Q_n/Q_0 ratio:

$$S_n^i = -\frac{Q_n}{Q_0} \sum_{j \in NU_n} p_{jn} \ln p_{jn} \; n = 1,\dots,N \qquad (6.23)$$

On the other hand, the outflow-based network entropy S^o is given by:

$$S^o = S_0^s + \sum_{n=1}^{N} S_n^o \qquad (6.24)$$

in which the first term is the entropy of the finite scheme defined for the supplies in Eq.6.17:

$$S_0^s = -\sum_{n=1}^{N} P_{0n} \ln P_{0n} \qquad (6.25)$$

and the second term is the conditional entropy of outflows including any demand, as defined in the finite scheme of Eq.6.20:

$$S_n^o = -\frac{Q_n}{Q_0} \sum_{k \in ND_n} p_{nk} \ln p_{nk} ; n = 1,\dots,N \qquad (6.26)$$

It is important to realise that, in either case of the inflow- or outflow-based expressions, what the entropy is measuring is the uncertainty associated with, respectively, the flow-joining and the flow-splitting processes. In other words, the entropy expressions are assessing respectively, the uncertainty in the origin of any flow entering the node, and the uncertainty in the destination of any flow leaving it.

That is the reason why the outflow-based formula, measuring the flow-splitting uncertainty, includes a special term for source flows. Imagining that all source flows in the network ultimately originate in one common super-source, as in the figure below, there may be uncertainty generated by the proportion of total supply that enters the network at each real supply point. That is modelled as uncertainty in the imaginary flow-splitting process at the super-source, hence the term S_0^s in Eq.6.24. Conversely, the uncertainty of the flow-joining process at an imaginary super-sink is in the basis of including the term S_0^d in the inflow-based formula, Eq.6.21.

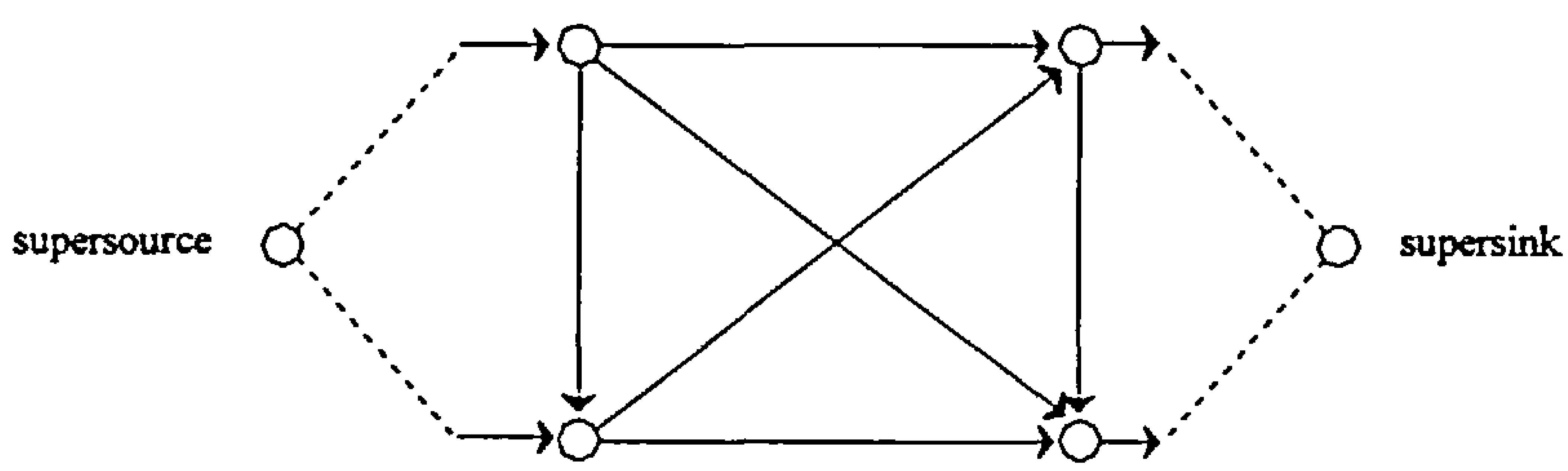


Fig.6.3 - Super-source and super-sink

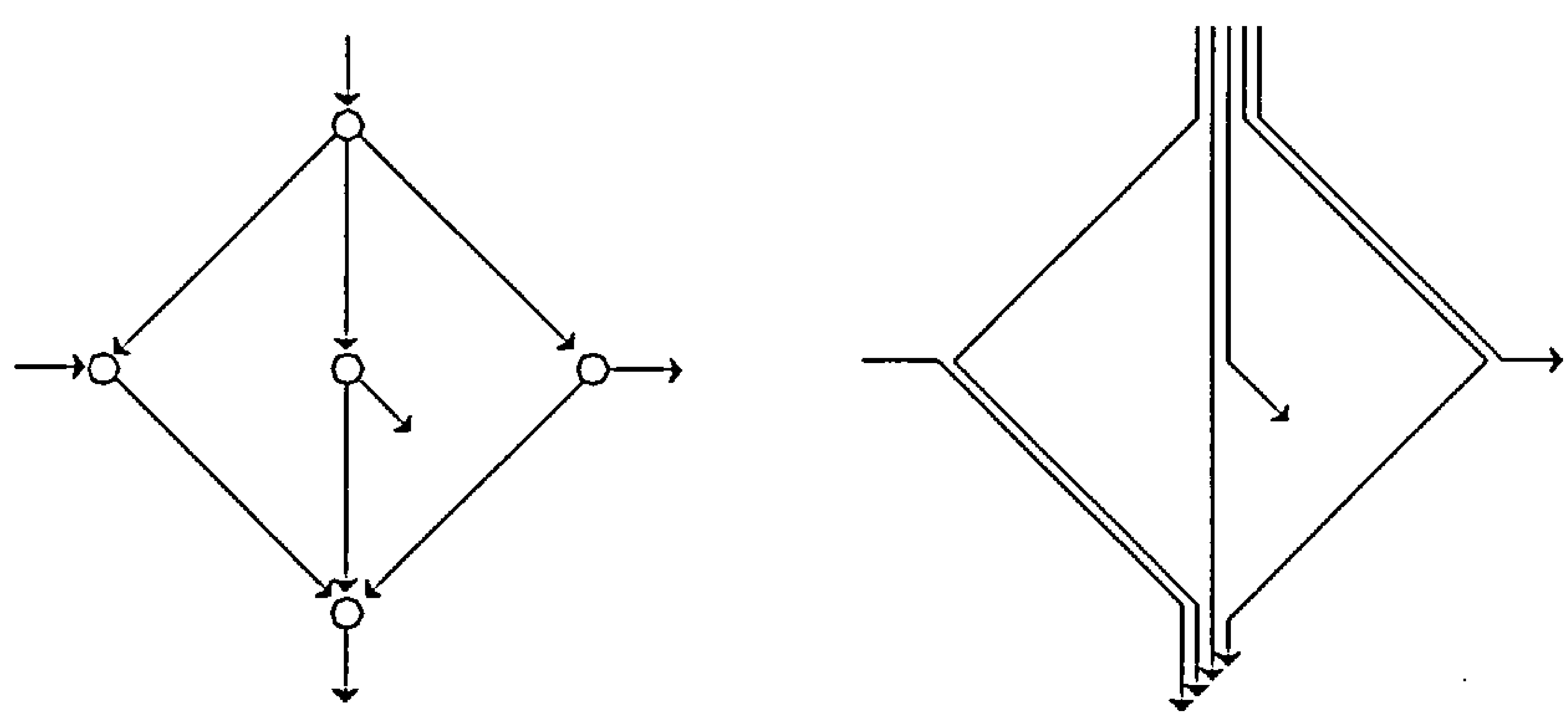
A less clear aspect of Tanyimboh's formulation for the nodal entropies, Eqs.6.23 and 6.26, concerns the weighting ratios that are used, Q_n/Q_0. These quantities are included as a consequence of deriving a purely nodal-based network entropy scheme. As mentioned before, the elementary entropy values at the nodes are calculated as the entropy of the flow-splitting/ flow-joining process, conditional upon the probabilities that flow passes/ reaches node n, Q_n/Q_0. However, it must be pointed out that those quantities do not constitute a finite scheme. Their inclusion in the derivation of the entropy expressions (Eqs.6.21 and 6.24) through the compound scheme entropy formulation (Eq.6.6) may be less than straightforward to justify in the face of Shannon's theory.

The need to include those quantities stems from the choice of finite schemes for Tanyimboh's formulation. It is possible, however, to formulate the very same problem in a different way in order to comply more clearly and with greater simplicity with Shannon's entropy principles. That is carried out in the following sub-section.

6.3.4. Proposed formulation

It has been mentioned before that the most uniform or least biased flow distribution would in principle correspond to the highest possible value of an entropy function measuring the uncertainty generated by the variety of different paths of supply to each and every node of a network. Rather than working with nodal quantities, a better way of reflecting that uncertainty is to define a finite scheme and a corresponding entropy function based on those paths. This, however, must be done without resorting to exhaustive path enumeration, a prohibitively heavy process for anything but the smallest networks.

As mentioned before, the universe of all possible paths in a distribution network (Fig.6.4a and Fig.6.4b) is the set of events whose uncertainty is to be measured. Together with the exhaustive and mutually exclusive probabilities associated with each path, they constitute a finite probability scheme.



(a) (b)

Fig.6.4 - Network paths

The uncertainty associated with the paths is ultimately generated by the fact that the flows carried by the various pipes of the network, as well as external inflows (supplies) and outflows (demands) are joined and/or split at junction nodes in an uncertain manner. Consequently, as indeed introduced by Tanyimboh, there will be two different ways of looking at that uncertainty, and two corresponding entropy functions: based on the flow-splitting and on the flow-joining processes.

The following procedure will be shown for the flow-splitting entropy, to begin with (the demonstration for the flow joining entropy is the exact reverse). In order to be able to express the entropy of the network without explicitly enumerating all the paths, a sequential process will be used that takes advantage of the following feature of distribution networks: there is at least one node where no link flows originate, i.e., which has only contributing links and demand flow. Such a *terminal* node is illustrated in Fig.6.5 (as well as its counterpart, in terms of deriving the flow-joining entropy formulation, the *initial* node).

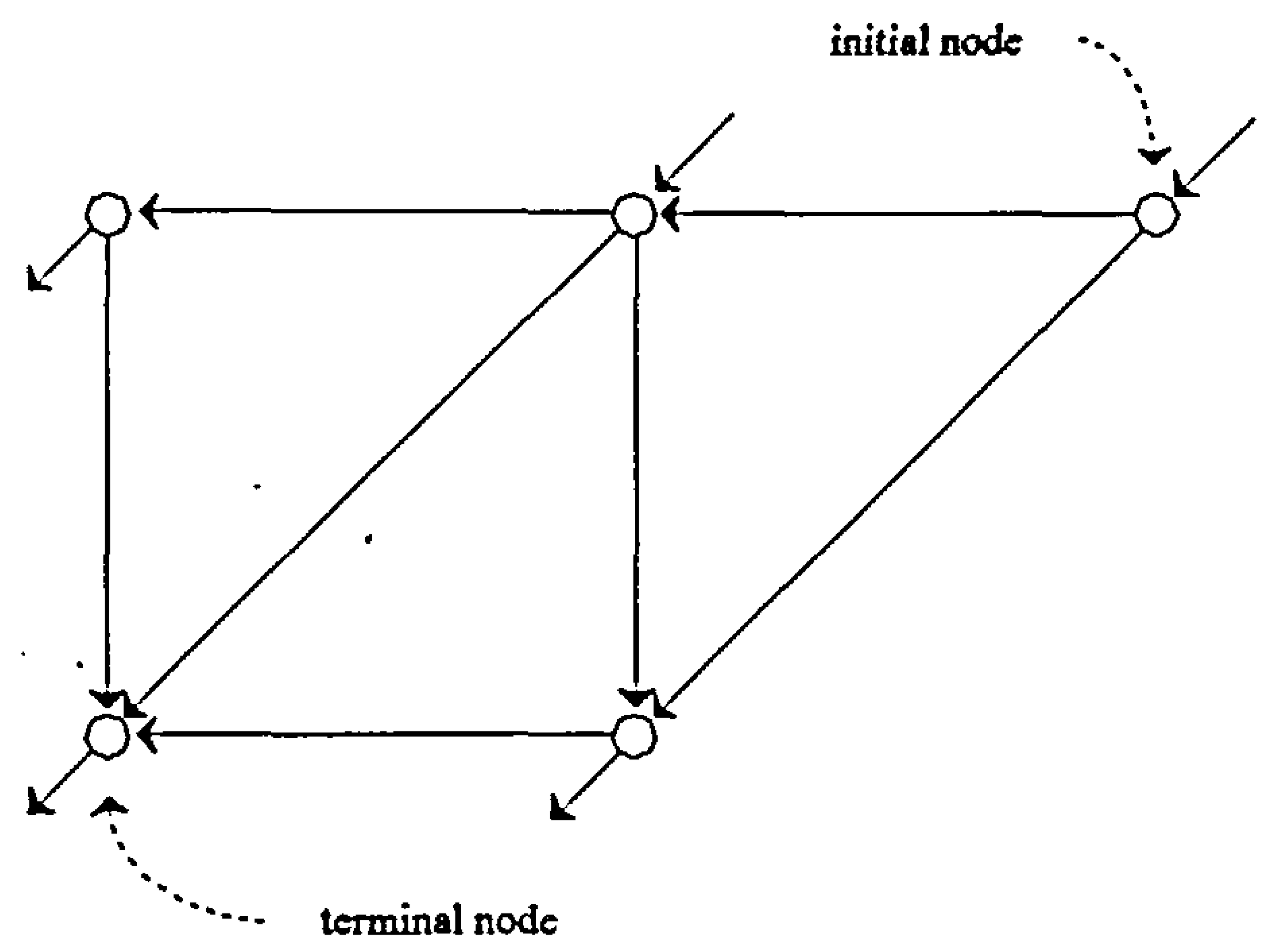


Fig.6.5 - Terminal and initial nodes

The entropy of the flow-splitting processes at such a terminal node is zero, as there is no splitting of flows at such node. This property is used as a starting condition for the following recurring expression for the "path" entropy of a network. The entropy of the flow-splitting processes that occur in all the paths downstream of any particular node n, S_n^{FS}, is given by the following expression:

$$S_n^{FS} = \sum_{k \in D^n} (-p_{nk} \ln p_{nk} + p_{nk} S_k); \forall n \in N \tag{6.27}$$

D^n being the set of downstream nodes from n, including the super-sink (p_{nk} therefore include p_{n0} as in the previous use of this notation), with $p_{nk} = q_{nk} / Q_n$. This expression basically calculates the entropy generated at the node by the flow-splitting process, and adds to it the entropies already calculated at the nodes that are immediately downstream in all the paths leading *from* it, appropriately multiplied by their respective probabilities (as calculated from the point of view of the alternative paths that emerge from n). This formula is applied recursively, starting at the terminal node(s) as mentioned, and proceeding to any node immediately upstream whose set D of nodes to which it contributes have all been calculated. Adopting the following notation:

$$S_{nk} = -p_{nk} \ln p_{nk} \tag{6.28}$$

The recursive expression Eq.6.27 can be written as:

$$S_n^{FS} = \sum_{k \in D^n} (S_{nk} + p_{nk} S_k); \; \forall n \in N \tag{6.29}$$

The sequential method thus established, easily implemented in a computer program, is guaranteed to cover all the paths existing in the network, and furthermore, to actually calculate the entropy S_n^{FS} associated with the uncertainty generated at the flow-splitting processes. For the generic node n, this quantity represents the uncertainty generated by the diversity of supply paths emanating from that node. The process is carried all through the network to the last initial node in order to yield the network entropy, or all the way to the super-source (see Fig.6.3) in order to include the entropy associated with the supply flows.

The entropy of the flow-joining processes S_n^{FJ} can be calculated by a similar process. For the generic node n, this quantity will now measure the uncertainty generated by the diversity of supply paths reaching that node. Beginning at the initial node(s) and using the complementary expression to Eq.6.29:

$$S_n^{FJ} = \sum_{j \in U^n} (S_{jn} + p_{jn} S_j); \forall n \in N \tag{6.30}$$

Where:

$$S_{nk} = -p_{nk} \ln p_{nk} \tag{6.31}$$

Having formulated entropy based on path uncertainty and an appropriate path-based finite scheme, it can be shown that the expressions developed by Tanyimboh for the total entropy of the network actually produce the same terms as those included in the full development of S_n^{FS} or S_n^{FJ} as given by Eq.6.29 and 6.30. This is not surprising in itself, as there was no reason to believe that the former would be incorrect, but merely that the way in which it was derived might not entirely comply with the principles of the entropy theory. The formulation proposed herein, having been developed for a different probability space and, it is believed, fully within the necessary requirements for the application of Shannon's expression, will if nothing else validate it definitely.

The equivalence of results between Tanyimboh's formulation and this work's proposed distinct alternative is best illustrated with an example, which also serves as demonstration of the method itself. Considering the multiple-source, multiple-sink, multiple-loop example network in Fig.6.6, the entropy of the flow-splitting processes in the network can be written from the

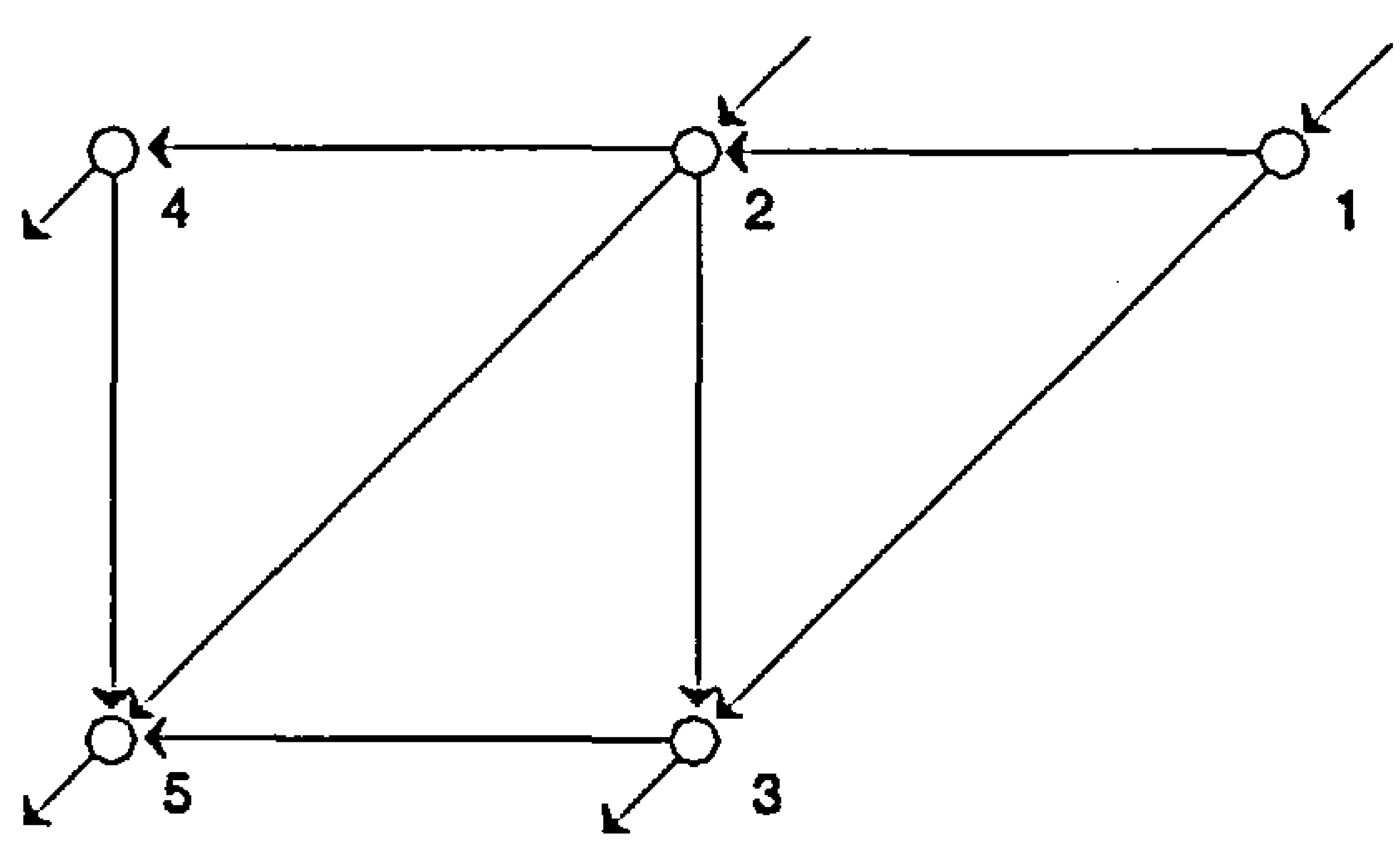


Fig.6.6 - Example network

super-source:

$$S_0^{FS} = S_0 = S_{01} + S_{02} + p_{01}S_1 + p_{02}S_2 \tag{6.32}$$

Expanding the terms:

$$\begin{aligned} S_0^{FS} &= S_{01} + S_{02} + p_{01}(S_{12} + S_{13} + p_{12}S_2 + p_{13}S_3) + p_{02}(S_{23} + S_{24} + S_{25} + p_{23}S_3 + p_{24}S_4 + \underbrace{p_{25}S_5}_{=0}) = \\ &= S_{01} + S_{02} + p_{01}\left[S_{12} + S_{13} + p_{12}(S_{23} + S_{24} + S_{25} + p_{23}S_3 + p_{24}S_4) + p_{13}S_3\right] + \\ &\qquad + p_{02}\left[S_{23} + S_{24} + S_{25} + p_{23}S_3 + p_{24}S_4\right] \end{aligned} \tag{6.33}$$

In which:

$$S_3 = S_{30} + S_{35} + \underbrace{p_{35}S_5}_{=0};\ S_4 = S_{40} + S_{45} + \underbrace{p_{45}S_5}_{=0};\ S_5 = 0 \tag{6.34}$$

Expanding the products and regrouping on the link and nodal entropy terms:

$$\begin{aligned} S_0^{FS} &= S_{01} + S_{02} + p_{01}(S_{12} + S_{13}) + (p_{01}p_{12} + p_{02})(S_{23} + S_{24} + S_{25}) + \\ &+ \left[p_{01}(p_{12}p_{23} + p_{13}) + p_{02}p_{23}\right](S_{30} + S_{35}) + \left[p_{01}(p_{12}p_{24}) + p_{02}p_{24}\right](S_{40} + S_{45}) \end{aligned} \tag{6.35}$$

Considering that $q_{01} = Q_1; q_{02} + q_{12} = Q_2$; etc., this expression can be written in terms of the probabilities $\overline{p}_n = \dfrac{Q_n}{Q_0}$ as used by Tanyimboh:

$$S_0^{FS} = (S_{01} + S_{02}) + \overline{p}_1(S_{12} + S_{13}) + \overline{p}_2(S_{23} + S_{24} + S_{25}) + \overline{p}_3(S_{30} + S_{35}) + \overline{p}_4(S_{40} + S_{45}) \tag{6.36}$$

This expression yields exactly the same terms as Tanyimboh's flow-splitting entropy expression, Eq.6.24:

$$S^o = \sum_{n=1}^{N} S_{0n} + \sum_{n=1}^{N}\left(\frac{Q_n}{Q_0}\sum_{k=1}^{ND_n} S_{nk}\right) = \sum_{n=1}^{N} S_{0n} + \sum_{n=1}^{N}\left(\overline{p}_n\sum_{k=1}^{ND_n} S_{nk}\right)$$

It must be noted, though, that the equivalence for the flow-splitting formulations is only verified between Eq.6.24, inclusive of source flow entropy, and Eq.6.29 calculated at the super-source, or between Eq.6.24, exclusive of source flow entropy, and Eq.6.29 calculated at the last initial node (the same would hold for the flow-joining formulation). It is important to realise that the nodal entropies calculated through Tanyimboh's formulation, Eqs.6.23/6.26 do not correspond to those calculated through Eq.6.29/6.30 at any given node. The former nodal entropies measure the uncertainty associated with the multiplicity of *links* leaving or entering the node, weighted by a factor that attempts to reflect the probability that flow will reach that node. The latter explicitly measure the actual uncertainty associated with the multiplicity of *paths* originating in, or reaching, the given node. Although both formulations yield the same global value for the network, the significance of the nodal entropy quantities is in fact quite distinct and it is believed that the path-based values relate more directly to the concepts of reliability of supply from or to a given node.

~ ~ ~

Some results of the presented formulation are worth discussing. Tanyimboh shows that, for any given network, the total entropy of the flow splitting processes (i.e. including the supplies at the super-source) equals the total entropy of the flow-joining processes (including the demands at the super-sink). That is certainly true for the formulation developed here. Also, if all the flows in Fig.6.6 are reversed, including source and demand flows, the total entropy of the network would not be expected to change. If the flow-joining expression is applied to the reversed network, the terms generated are obviously the same as in Eq.6.36.

Of more importance to a correct understanding of the way the different terms relate in the formulation is the case of tree-type networks such as illustrated in Fig.6.7. Such a network should have no entropy associated with flow-joining processes because in effect there are none, except for the term referring to the demands at the super-sink. That means that, for the same set of demands, any tree-type layout has the same flow-joining entropy, or entropy of supply to a node. Also, any tree-type network portion has zero flow-joining entropy.

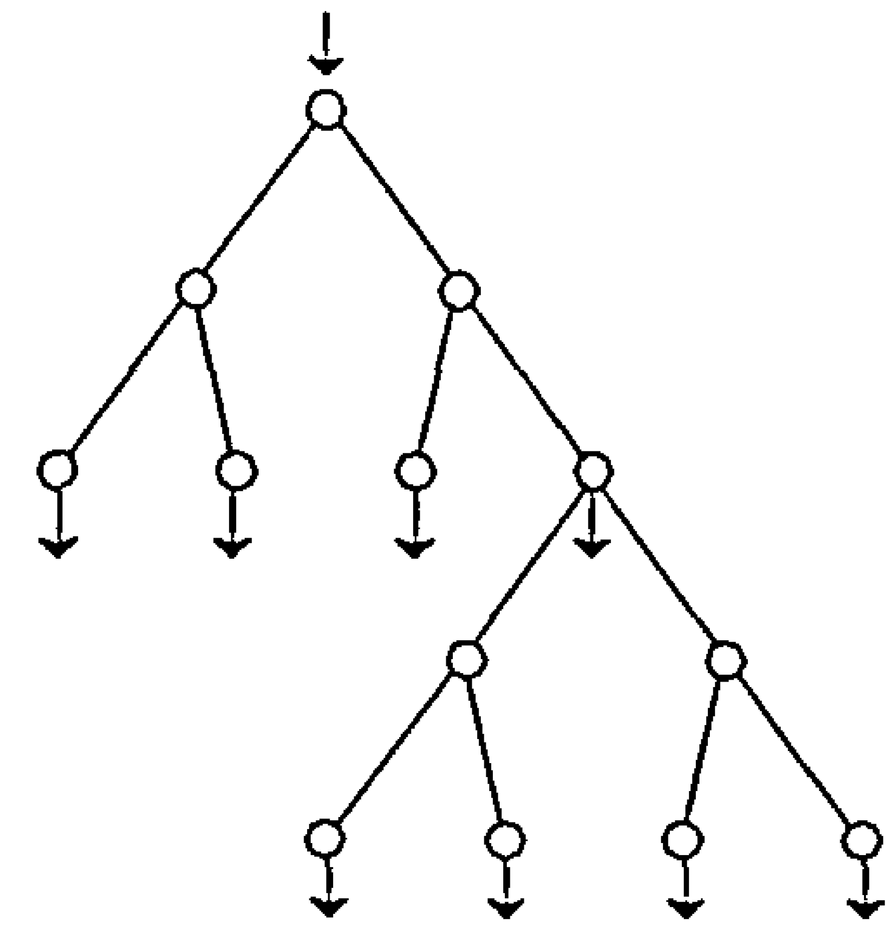


Fig.6.7 - Tree-type network

In the case of Fig.6.7, making all demands equal to one unit of flow, the only non-zero terms in the expression for S_n^{FJ} are due the flow-joining entropies of the demands, which yield a value of 2.0794. On the other hand, there is uncertainty relative to the flow splitting processes (but not to the corresponding term for source flows generated in the super-source). In fact, the flow-splitting network entropy S_n^{FS}, as given by Eq.6.29, produces the same result: 2.0794. This illustrates an important property of the entropy functions developed, namely that the total network entropy is the same calculated either way. It also shows that tree-type networks have zero uncertainty of supply as there are no flow-joining processes, but may have non-zero flow-splitting entropy as there is uncertainty associated with the destination of any flow circulating through a node leading up to more than one path.

6.3.5. Maximum entropy flows as a measure of reliability

Having modelled network flows as probabilities and reached a formulation for the entropy of network flows, inferring least biased estimates for the values of the link flows of a general network can now be formulated in the classic fashion as an entropy maximisation problem subject to flow equilibrium. Given the above mentioned equivalence, in terms of the global network entropy, both S^{FS}, S^o or S^{FJ}, S^i may be used. Formulating on S^{FS} :

$$Max_{(q)} S_n^{FS} = \sum_{k \in D^n} \left(-\frac{q_{nk}}{Q_n} \ln \frac{q_{nk}}{Q_n} + p_{nk} S_k\right); \forall n \in N \tag{6.37}$$

which is maximised on q subject to the continuity equations at the nodes:

$$\sum_{j \in U^n} q_{jn} = \sum_{k \in D^n} q_{nk}; \forall n \in N \tag{6.38}$$

and to the normality conditions on p_{nk}:

$$\sum_{k \in ND_n} p_{nk} = 1 \Leftrightarrow \sum_{k \in ND_n} q_{nk} = Q_n; \forall n \in N \tag{6.39}$$

as well as the non-negativity of the decision variables q:

$$q_{nk} \geq 0; \forall n \in N, \forall k \in D^n \tag{6.40}$$

This is a constrained non-linear programming problem with a unique global maximum which can be solved by any appropriate technique. Tanyimboh (1993) may be consulted for a detailed discussion of the subject, which is tackled by that author with the purpose of developing an entropy-based approach to the design of water distribution networks.

Network flow entropy is one of the most attractive methods available for providing a quantifiable and easily calculated indirect reliability measure which can be used for the purpose of engineering analysis of the network. Combined with the maximum entropy flow distribution, which provides a reference value of entropy as the optimum against which all possible flow distributions can be measured, it provides a tool that very much satisfies the requirements set out in the introduction to this section, and indeed the broader requirements of the performance evaluation analysis.

Of particular interest to the present work is the result demonstrated by Tanyimboh that, on a nodal basis, maximum entropy flows for single source networks correspond to the uniform distribution U applied to the finite probability scheme used: each demand node should receive an equal proportion of its demand from each of the contributing paths to the node. That property is used to develop a simple but efficient algorithm for calculating maximum entropy flows in single source networks, which works its way from downstream terminal nodes to upstream initial nodes, by splitting the flow at each node in equal parts throughout its contributing paths. Tanyimboh uses a simple node weighting technique to avoid explicit path enumeration. The method, which is summarised in Appendix C, is easily programmable, is not iterative or complicated by numerical optimisation, and furthermore, the system of equations for flow equilibrium does not have to be explicitly solved. It is important to notice, as pointed out in Appendix C, that this method corresponds directly to the optimisation of the path-based formulations as described above.

Tanyimboh mentions that an advantage of this approach to finding the maximum entropy flows is avoiding the problem of whether to include the demand-generated entropy, by relying on those values as deterministic quantities. Conversely, the use of the source flows as equally deterministic quantities would introduce the obvious disadvantage of restricting the method to single source networks.

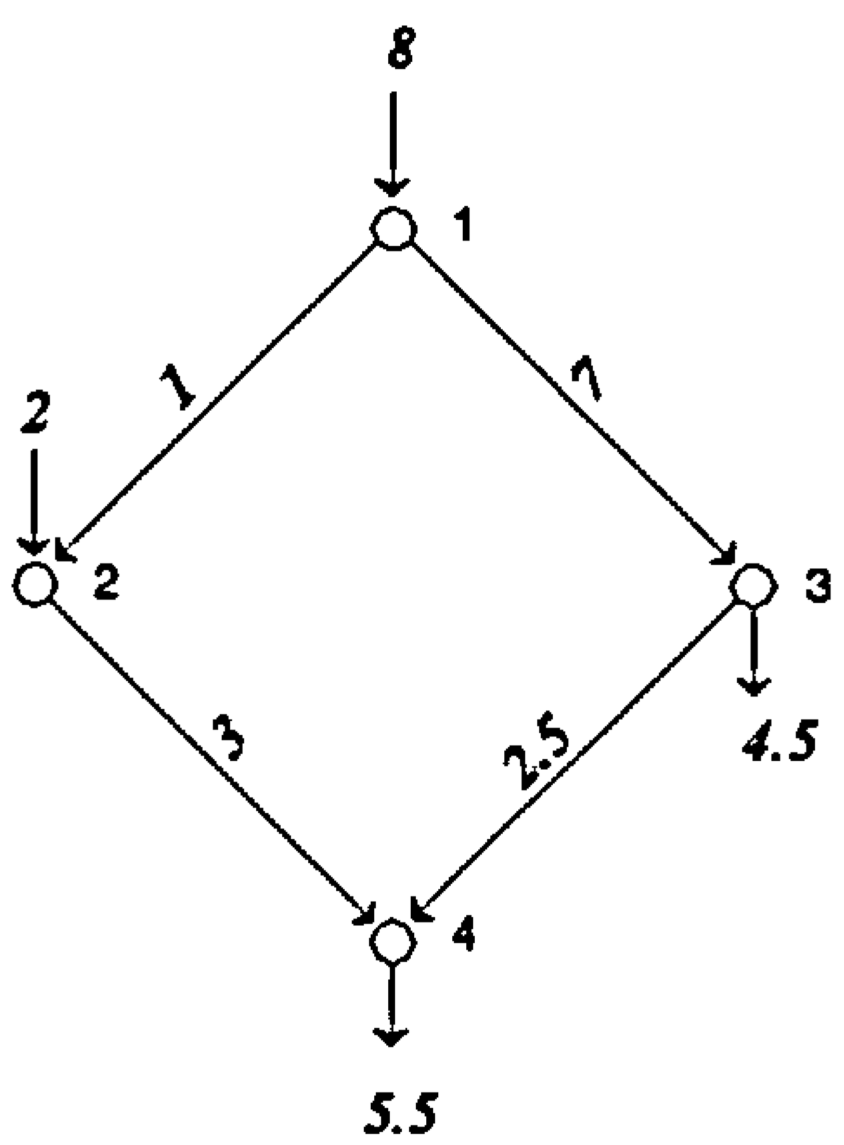
In effect, none of those observations need necessarily hold, depending on what the calculation is intended as. In fact, the analysis of maximum entropy flows in networks may, perhaps should, be looking for the most even distribution of source *or*[6] demand flows, and a super-source or super-sink node may be used in order to circumvent the problem. That is, the calculation can be carried out for the equivalent network where all the supplies are connected by imaginary links to a super-source or all the demands are connected by imaginary links to a super-sink, as represented in Fig.6.3.

Particularly in the case of sources, that would allow for the study of multiple-source networks, in which the values of the several source contributions would also be subject to the optimisation. If the purpose of the exercise is to determine the best possible flow distribution in terms of reliability, the inclusion of those terms in the optimisation would simply give us a measure of the redundancy levels of their respective actual values.

---

6 (exclusive *or*)

In fact, the algorithm developed by Tanyimboh is perfectly valid for multiple-source networks where all the sources in the maximum entropy solution contribute the same amount to each and every supply path to the demand nodes[7]. The single source network is a particular instance of this. If it is accepted that a network is demand-driven (and the analysis of water supply reliability is primarily focused on guaranteeing the satisfaction of those demands), it may be equally accepted that the maximum entropy flow distribution will include the specification of maximum entropy source flows as the ideal reliability solution to be used as a comparison standard. In Appendix C the appropriate modification to the algorithm is shown.



Node	S^{FJ}	$MaxS^{FJ}$	$S^{FJ}/MaxS^{FJ}$	S^{FS}	$MaxS^{FS}$	$S^{FS}/MaxS^{FS}$
1	.000	.000	.000	.947	.999	.948
2	.637	.693	.918	.000	.000	.000
3	.000	.000	.000	.652	.602	1.083
4	1.036	1.099	.943	.000	.000	.000
GLOBAL	1.258	1.292	.973	1.258	1.292	.973

Fig.6.8 - Example network for maximum entropy flows calculation

The example in Fig.6.8 shows a simple multiple-source, multiple-demand network with a given flow distribution, which is compared in the table to the maximum entropy flow distribution calculated using the method presented in Appendix C (including source flows maximisation).

7 This does not necessarily make all source supplies equal.

The node table shows the path entropy values both for the supply paths *to* the node (flow-joining entropy or S^{FJ}) and for the demand paths *from* the node (flow-splitting entropy or S^{FS}), for the actual flows as well as their respective maximum values. The ratios between the two are also calculated: $\frac{S^{FJ}}{S^{FJ}_{\max}}$ and $\frac{S^{FS}}{S^{FS}_{\max}}$.

It must be noted that the "global" values represent the whole of the network and are therefore calculated at the appropriate super-node (super-source for S^{FS} and super-sink for S^{FJ}).

The maximum path entropies are obtained from the maximum entropy flows and are the values that maximise the appropriate path-based entropy expression (Eq.6.29 or 6.30) written at the respective super-node. It should be noted that this does not necessarily correspond to the highest possible entropy value at each node, as a non-optimal flow distribution may occasionally generate a higher entropy at a particular node than the maximum entropy flow distribution. The example illustrates this property, for which the maximum entropy flow distribution produces a lower flow splitting entropy at node 3 than the actual flows.

The ratio between actual and maximum entropy was calculated in order to provide an idea of how close the actual values are to fulfilling the redundancy potential of the network. It can be seen, for example, that there is no uncertainty as to diversity of supply paths to node 3, but there is uncertainty of demand paths from node 3. Likewise, there is no doubt where the flow that reaches node 1 is coming from: there is a single supply path, hence no uncertainty – and equally, no redundancy.

6.4. RELIABILITY PERFORMANCE EVALUATION

6.4.1. Introduction

In similar fashion to previous domains, one of the objectives of this chapter is to analyse the applicability of the standardised performance assessment framework to the field of reliability of distribution networks. As seen before, the performance evaluation framework establishes three types of entities for each network property or behavioural aspect it analyses: (i) A relevant state variable, that is, the quantity which is chosen to represent reliability at the network element level; (ii) a penalty function, mapping the values of the state variable against a scale of index values; and (iii) a generalising function, used for extending the element-level calculation across the network, producing zonal or network-wide values.

The first step of the process, and the one that seems the most critical for this performance domain, has been dissected in the previous sections of this chapter. It has been discussed in sections 6.2.4 and 6.3.1 that for the purposes of a quantifiable performance assessment method, it is appropriate to associate reliability of a network with its redundancy of supply, a property for which an entropy-based measure has been developed that would appear to be a good candidate to fit the above scheme.

The detailed analysis of reliability concepts and measures has shown that this is a domain where the performance evaluation system as described above may not find the applicability it has shown in other areas. As mentioned in the introduction to this chapter, reliability is a field where the concept of performance must be viewed in a different light, closer to a potential property of the networks than to an operational-style characteristic that can be shaped or changed with ease.

The following text will deal with those subjects, in particular discussing the variables that may be interesting to analyse in reliability related subjects. The relationship between entropy as a reliability measure, and the various types of reliability that may be interesting to measure in a performance analysis of the network, is debated. The possibility of defining penalty curves and generalising operators and of drawing system graphs is discussed and illustrated.

6.4.2. Reliability, entropy and performance

It has been mentioned in previous sections that the existence of backup or alternate supply pathways constitutes redundancy in the distribution system. Reliability, in its most general sense, defines how well the system performs in meeting the demands upon it. It can therefore be said that redundancy, as it represents the presence of alternate supply paths through the network, and the associated ability to maintain adequate service with some components out of service, is a major contributor to reliability.

It has been seen that the maximum entropy flow distribution corresponds to the most uniform distribution of flow among all the paths of the network, in the sense that all the alternative paths of supply to a node (or of demand from a node) carry equal fractions of the demand (or supply) at that node. There is every reason to believe that this distribution will take advantage of the diversity of paths existing in a network in the best way. From the point of view of supplying demands, that diversity is of course only present as long as the network is not purely tree-type, as it has been seen before. Tree-type networks have zero diversity, hence zero entropy of supply paths to a demand node. Tanyimboh (1993) shows that, if all the tree-type type layouts that are possible to define within a general network are considered for their flow distributions, the maximum entropy flow distribution is the one lying furthest away into the solution space of the optimisation problem from the tree-type solutions.

The maximum entropy flow distribution is the most central, or most uniform, of all the distributions capable of satisfying the demands of the network. Indeed, Tanyimboh has worked on optimal design of networks based on the assumption that the magnitude of the possible changes to the values of the link flows of the network can be minimised by designing the network to carry maximum entropy flows. These possible changes always carry a heavy hydraulic penalty, as the headloss varies approximately with the square of the flow rate. Consequently, the closer the flow distribution is to a mid-point, the smaller those differences squared will be, and the smaller the 'wasted' headloss. An additional consideration is that, if those headlosses are minimised in such a way, then the greater will be the capability to withstand flows caused by exceptional or unexpected demands, should they happen.

It is important however to recall that entropy-based measures do not effectively translate the hydraulics of the network, and as such are limited as a measure of redundancy to a mainly topological role.

So far, reliability is a concept primarily associated with the continuity of supply and (or) the satisfaction of pressure and other hydraulic related requirements. It is therefore mainly concerned with network element failure or malfunction, or exceptional demand situations, as the main threats to the stability of those requirements.

However, it might be useful to recall at this stage what the primary objectives of good design, operation and management of water distribution systems are: to supply sufficient quantities of potable water, at adequate pressure and at minimum possible cost. The reliability performance of a water network should therefore be seen from a broader range of perspectives than the *mere* satisfaction of hydraulic or continuity-of-supply requirements. There are other objectives which would benefit from the introduction or evaluation of reliability, namely those that have to do with the other main area of operational performance, water quality. Domains such as the propagation of a desired substance (e.g., chlorine residual or fluoride) and source utilisation (particularly as regards the influence of a good quality water over a less good one, or the impact of a pollution incident) deserve to be looked at from the point of view of how reliable is the network likely to be, relative to the established objectives in each one of them.

It is in the face of those several, sometimes conflicting, reliability requirements that the path-based formulations developed in the previous section may come to useful application. It will be recalled that S_n^{FS} or S_n^{FJ}, as given by Eq.6.29 and 6.30 respectively, measure the uncertainty in the diversity of paths originating from, or leading up to, a particular node of the network. It would appear then that those two measures can be associated with different objectives in reliability analysis. The classical problem which consists of trying to enhance the capability of the network to reach a given demand node can be looked at from the path-based flow-joining entropy. On the other hand, the flow-splitting analysis which measures the diversity of paths emanating from a given node is better suited, for example, to the situation of trying to protect the area of influence of a given node. In either case, the path-based formulation provides the advantage, over previous entropy expressions, of nodal entropy values which have real network reliability significance.

The performance of a water distribution network is, from the reliability point of view, markedly different from the other fields analysed. It has been mentioned before that reliability is not an operational property of the network, which can be easily changed or adjusted with operation practice. Reliability is fundamentally conditioned by the shape and layout of the network. Measuring reliability for the purpose of assessing the performance of a network cannot hope to have the same potential for comparison as some of the other measures analysed previously. It is difficult to compare different networks between themselves, by classifying them according to some universal referencing system. The best that can be hoped for, then, is to be able to analyse and compare different layouts to supply the same distribution of demand points, or to compare different ways of utilising the same layout to supply the same demand points. For both these objectives, the entropy measure does seem to be the best candidate to provide useful results.

All in all, the use of entropy-based measures does carry some benefits in that respect, namely the fact that it is a dimensionless measurement – two networks with the same layout and flow directions, and with different but proportional demands and flows, will produce the same entropy values. Also, the fact that for each layout it is possible to define the optimal flow distribution according to an entropy maximisation criterion, provides both a way of grading any given flow distribution against that ideal, for a fixed layout, or the comparison of different layouts based on the optimal entropy values that each can achieve. Either is a fair means of comparison, and a suitable approach to the development of performance measures.

For those reasons, it seems appropriate to choose as performance evaluation variables for reliability/ redundancy the ratios introduced in the previous section, which pitch the entropy of flow distribution against the corresponding maximum possible (optimal) value, on a nodal or global basis. Two measures will thus be obtained as follows:

(i) entropy of supply paths to the node as a percentage of the network's optimal potential, $\frac{S^{FJ}}{S^{FJ}_{\max}}$;

(ii) entropy of demand paths from the node as a percentage of the network's optimal potential, $\frac{S^{FS}}{S^{FS}_{\max}}$;

6.4.3. Penalty curves and generalising functions

The performance evaluation system that has been applied elsewhere in this work must be carefully analysed for the purpose of applying it to the present domain. The measures proposed above are defined at nodal level.

The first observation concerns the use of penalty curves as defined previously for a range of levels of service, conventionally from 0, or no service, to 4, or optimum service. This is where the different nature of reliability performance, as compared with other types of performance, is perhaps most apparent. Unlike the other domains, it is not easy to establish in a definite manner what 'no service', 'inadequate service', 'adequate but not optimal service', etc., might correspond to in terms of the assessment method developed.

It is recalled as an example that for the hydraulic measures, those concepts have been associated with values of the performance curves in a meaningful manner, taking advantage of established requirements such as minimum pressure levels, etc.. That happens to such an extent that there is some degree, or the possibility thereof, of calibration between different performance measures.

In the water quality domain, the existence of very precise guideline values, normally as mandatory upper or lower bounds for the respective parameters, allows for the clear definition of criteria. The design of penalty curves is therefore relatively straightforward from the conceptual point of view, even though in matters of detail some margin is left to the analyst.

In the present case, none of those situations arises. Apart from the optimum, which could presumably be associated with the maximum entropy flow distribution, any other level of service definition in this domain is arguable. To be consistent with the adoption of a redundancy-based measure, it would seem logical that the 'no-service' situation corresponds to the zero path diversity, or zero entropy, generated at nodes with single supply or demand paths. For this convention to be adopted, it must be borne in mind that nodes that have no supply/ demand paths to/from them are effectively omitted from the network and are automatically left out (they do not contribute to the network entropy functions). Within these limitations, a possible penalty curve for $\frac{S^{FJ}}{S^{FJ}_{\max}}$ or $\frac{S^{FS}}{S^{FS}_{\max}}$ would be as in figure 6.9.

Since it is possible for nodal values to be greater than one (as seen in the previous section), the curve would be defined as optimal for values greater or equal than one. This feature is the only real difference between using this penalty curve and the actual value of the variable, given the direct proportionality. However, the use of the penalty curve has the advantage of bringing the possibility of standardisation achieved in other areas to this domain as well, and that is the main reason why it is introduced here.

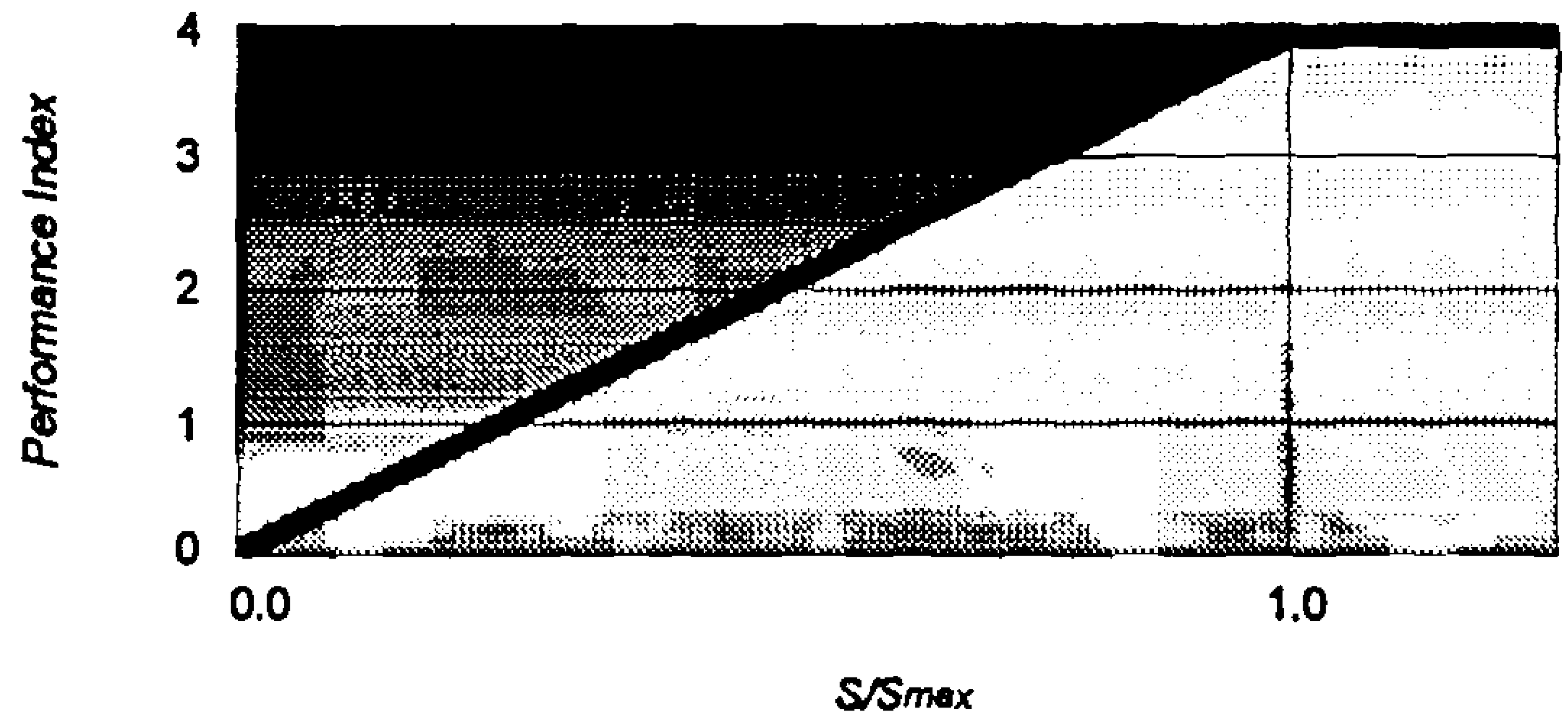


Fig.6.9 - Penalty curve for entropy measure

The problem of choosing a generalising function is in this case solved from the outset as the path-entropy expressions can be calculated for the whole network. The same penalty curve as above can be applied to the value of the variable $\frac{S^{FJ}}{S^{FJ}_{\max}}$ or $\frac{S^{FS}}{S^{FS}_{\max}}$ calculated for the whole network, which is a departure from the approach used previously of applying the generalising function to the nodal performance values. It makes better sense, in this case, to take advantage of the global entropy values, which, just as the remaining nodal values (it should not be forgotten that the network value corresponds to the super-node), have a precise and significant meaning in this case.

6.4.4. Application examples

The above penalty functions will now be tested using some of the networks presented in previous examples. Figure 6.10 shows the previously employed concept of a system graph, calculated for both flow-splitting and flow-joining measures, for test network 1 of Chapter 5.

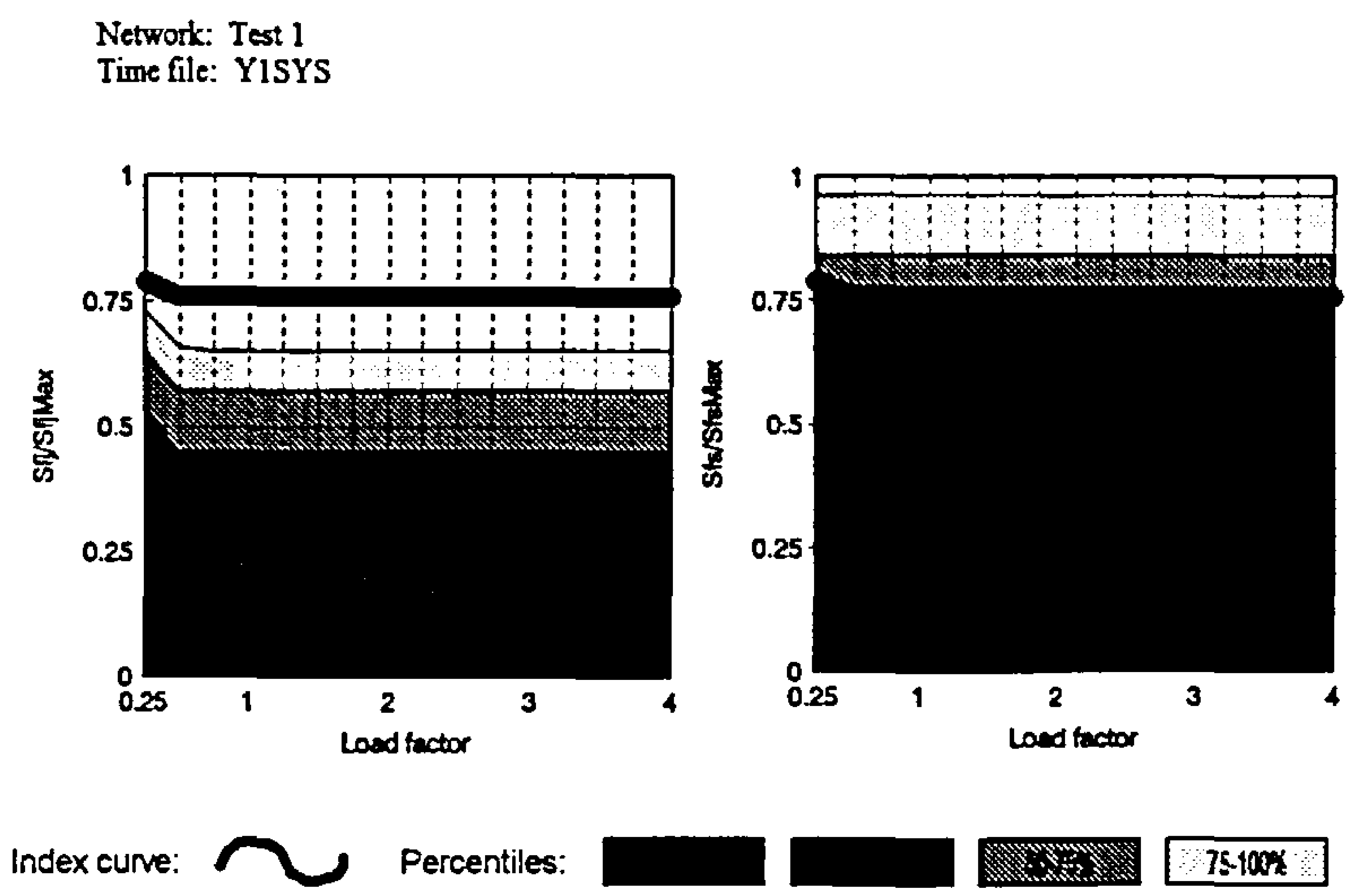


Fig.6.10 - System graphs for test network 1

The network seems to be operating at a reasonable fraction, around ¾, of its maximum redundancy potential, as given by both global entropy curves, similar as expected. The layout of this single-source network is reasonably symmetrical along its longitudinal axis (the diagonal defined by the reservoir and node 16, as seen in figure 5.13), and the pipe sizes favour a good spread of flow (Appendix A), which justifies the reasonable degree of uniformity of the flow solution.

This network has quite a proportion of its hydraulic capacity still left at normal operating conditions. As the demand loads increase, so do the flows, but the flow distributions are proportional given that capacity. For proportional flow and demand distributions on a similar layout, the entropy values will be constant, and that property is one of the most noticeable features of the above diagrams.

The initial demand load, which corresponds to 0.25 of the average load, generates very small flows around the network, in many cases tending to zero for the degree of numerical accuracy used in the simulation. These actually correspond to a more uniform flow distribution, hence the slightly higher values produced. The width of the dispersion bands shows that increase is greater for certain sets of nodes (certain areas) of the network, than for the global network value, which makes sense.

Despite the global curves being forcibly the same, it can be seen that the flow-splitting processes in the network paths themselves, thus at the network nodes, generate higher entropy levels, or fulfil a greater part of their redundancy potential, than the flow-joining ones. This is to be expected from any single-source, multiple-demand-point network, as the latter produce considerable more flow-joining entropy than the (zero) flow-splitting entropy generated at the former.

As the totals must be the same, the network must be responsible for the remainder in both cases, and must therefore generate more flow-splitting diversity, which is to say, more diversity of supply paths *from* its nodes. Some of the networks' nodes are actually very near the full potential in terms of redundancy, which can be seen on the graph on the right, whose topmost band almost reaches the value of 1.00. The flow distribution along the paths leaving those nodes is almost ideal.

The fact that the flow-splitting network entropy yields higher nodal values than the flow-joining ones, which will happen for most networks due to the fact that they usually have considerably more sinks than sources, actually has interesting reliability-related implications. The higher diversity of paths flowing from certain nodes will mean that those nodes influence other nodes in a more redundant way. Depending on the occurrence, this can be favourable or unfavourable. Supposing those nodes include various sources of water, one of which is more prone to periods of lower water quality supply, it will probably be desirable that that node will present less of an influence on the network, i.e., that it generates a smaller diversity of paths originating from it. The same would happen if a contamination accident is feared or actually occurs at a particular node.

Conversely, if a node is a desirable source, for water quality reasons or simply for hydraulic reasons – as in the classic reliability approach which looks for connectivity and the guarantee of supply – then it will be the other way around.

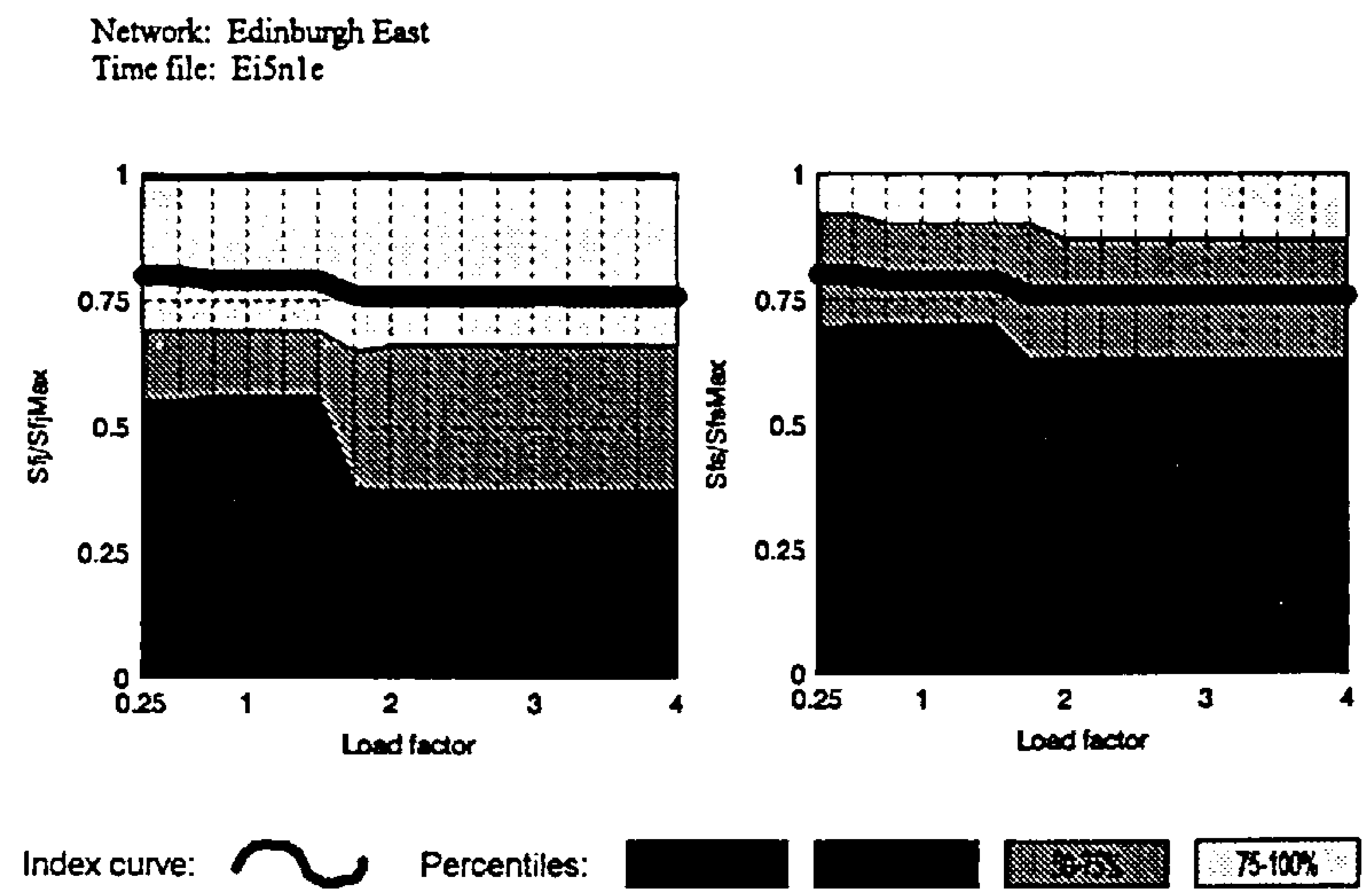


Fig.6.11 - System graphs for the East-Edinburgh network

A second example is shown in figure 6.11 and concerns a real network, previously introduced; the East-Edinburgh distribution system. This system has a greater spread of entropy values, which is to be expected since there are quite a few branched sections (see network schematic in Fig.5.18) generating zero flow-joining entropy, but also elongated and well looped sections which are probably operating near their potential. This example is shown however to illustrate a different use of the performance curves. It has been seen in the previous network that the simple variation of demand loads may not be very expressive, for the flow distribution will tend to vary proportionally, which yields the same entropy values, as seen before.

The same will happen for 24-hour, extended period simulations, which use an even narrower band of demand loads. For that reason, it is more meaningful and informative to study a range of operating conditions, which despite being based around the system graph philosophy, introduces operational changes that are likely to be employed to face those demand levels, or in need to be tested. This type of use is more in line with the exploratory, 'potential' nature of the entropy measure.

In the case of the Edinburgh network, a likely operational scenario is tested which corresponds to the introduction of lower pressure limits at the pressure reducing valves of nodes 163 and 173, with the objective of a leakage reduction exercise. The entropy levels of the network drop, as a consequence of the flow being concentrated in fewer paths around the affected areas

6.5. SUMMARY AND CONCLUSIONS

This chapter assesses the role of reliability in the performance of water distribution systems. It begins by reviewing the subject of reliability in water supply and distribution, with the objectives of the performance evaluation framework as much as possible in mind. Direct and indirect methods for assessing reliability are covered, and the latter concentrated upon given their greater suitability to the present analysis. It is shown that for the purposes of a quantifiable performance assessment method, reliability of a network may be equated to network redundancy, a property for which an entropy-based measure has been developed. Unlike the other two fields of study, it is not possible to define for reliability of water network the same type of clear-cut criteria that allow the performance evaluation framework introduced in Chapter 3 to be fully developed.

The above mentioned choice of reliability measure – network flow entropy – eventually adopted in this chapter is arguably the best available methodology for the purpose. It has been formulated, developed and manipulated in this chapter in order to provide the fundamental contribution of a quantifiable and flexible assessment of reliability. A new formulation is proposed which allows for the development of measures with network reliability significance at the nodal level – i.e., capable of being calculated for any node of the network, and relating to either incoming or outgoing flow – which were previously unavailable and are useful for the purpose of performance evaluation following the approach adopted in this work.

The applicability of the performance evaluation framework to reliability is then discussed and is found to be less straightforward than in the case of its hydraulic and water quality counterparts. A discussion of the relationships that it is possible to establish between the formulation of the entropy-based function and the variety of meanings that the concept of reliability may have in water supply is carried out. It is found that the expressions available to calculate reliability from a flow-splitting and a flow-joining uncertainty viewpoint help clarify what in reality is at stake when the concept of reliability is formulated against the conflicting objectives of, for example, ensuring continuity of supply and attempting to limit the effects of a pollution incident at source. Reliability is shown to have a broader meaning than that traditionally employed in water distribution engineering.

The main contributions of the work presented in this chapter are, apart from a comprehensive analysis of reliability measures of performance and the selection of a network flow entropy function as a quantitative measure with potential to integrate the current performance analysis, the development of an alternative, clearly derived formulation that clarifies, complements and validates the established method for the calculation of that function, offering the capability of network reliability measure at nodal as well as global level, both with precise redundancy-related meaning. The discussion of their properties and use for reliability evaluation in a broader sense than in the traditional approach is carried out with the help of illustrative examples, following the performance evaluation framework previously established.

CHAPTER 7

CONCLUSIONS

7.1. SUMMARY AND CONCLUSIONS OF PRESENT RESEARCH

7.1.1. General objectives

The present study proposes to analyse the performance of water distribution systems from an engineering point of view, with the objectives of developing a greater understanding of the subject and to define a methodology for direct support to the analysis and design procedures currently available to water engineers. Namely, the study sets out to assess the notion of performance in the various fields of water distribution systems and select those that may lend themselves to a technical approach; to develop a systematic and quantifiable approach to performance evaluation that may be used in direct support of water network engineering and operation, and test its applicability; and to analyse each field in detail and select the various elements that are necessary for the application of the performance evaluation approach.

Overall, the objectives of the study have been largely fulfilled. The performance assessment framework developed satisfies the requirements and criteria initially identified as essential. The main results achieved through the above steps were a crisper and deeper definition of the concepts of performance of water distribution networks for water engineering purposes, and their standardised quantification in a systematic manner. The immediate consequence of this is the possibility of reformulating the objectives traditionally employed in such water engineering tasks as the optimal design of new networks, expansion and rehabilitation of existing ones, and general operational control of distribution systems.

Three main performance areas in water distribution have been analysed with direct knowledge gains, including for two of those the effective development of new modelling tools. The performance assessment system was found to have the potential to drive any of the currently used engineering analysis and design tools, with particular relevance to the technical decision-making processes.

The present work constitutes a first comprehensive effort to analyse and quantify the technical performance of water networks. The improved definition and understanding of such performance is a result both at global level and in each of the fields analysed, in a innovative systematic way. The performance assessment framework provides a shift in the way engineering problems are formulated in water supply, allowing for greater control of analysis objectives and improved sensitivity. It is believed the system developed effectively addresses the problem of studying wide ranges of operating conditions in an unified and consistent approach.

The following sub-sections summarise and conclude upon each of the main steps followed in this study.

7.1.2. The development of a performance evaluation framework

A general methodology for the evaluation of technical performance of water supply and distribution systems is presented in Chapter 3. A systematisation of concepts helps establishing a standard approach to performance assessment that will be used as the common tool throughout the remaining chapters in this work. Standardisation is especially desirable in order to bring to the same quantified basis the various aspects that will be considered.

The method consists of choosing a state variable or network characteristic that quantifies the aspect relative to which performance is being assessed; a grading of the performance according to that variable; and a generalising function. The grading is translated through flexible penalty curves, which score the working range of values of the given state variable against a conventionalised system of performance gradings. The penalty curves are as much vehicles for common sense and level of service policy criteria as for the analyst's or engineer's sensitivity to a given aspect of a water network's behaviour.

The objectives of flexibility, standardisation and suitability for computational application through a numerical, quantitative approach are achieved. It is important, however, to bear in mind that the choice of state variable, indeed of performance measure, must above all contemplate network properties that can be standardised for the purpose of comparison, if not between different networks, at least between different operational demand scenarios.

The accuracy of the method is ultimately inherited from the source of the data it uses — mostly simulation results — and, conversely, its role can be seen as a synthetic analysis tool to avoid time-consuming examination of those data. The methodology has the potential to drive any of the currently used analysis and design processes, and was designed to be easily included in a simulation model.

The following chapters applied this methodology to the three main areas of water system technical performance: hydraulics, water quality and reliability. The systematic approach presented here is followed, with the identification of key aspects, their respective decision variables, the setting up of penalty curves and generalising functions, and the analysis of the resulting graphs for a variety of case studies.

7.1.4. Hydraulic performance in water distribution networks

Hydraulic performance was the first area to be analysed in the light of the proposed methodology. The assessment of hydraulic performance in water supply systems is an increasingly important topic in an industry progressively driven by a need to deliver competent levels of service. Conversely, the tools and procedures used by designers and engineers are based on more simplistic criteria than those implied by the current growing concern over all aspects of the networks' behaviour.

In Chapter 4, a brief overview of hydraulic modelling tools in use is given, as a basis upon which the whole analysis is to be built. The selection of what hydraulic state variables to include in a performance evaluation system is then presented and measures concerning pressure, pressure fluctuation, flow velocity and energy consumption are proposed. The corresponding penalty curves and generalising functions are discussed, as well as the suitability of the various measures to the framework proposed and to the objectives of the work.

Illustrative examples are given and the use and potential of the methodology explored. The results presented for some realistic case studies clearly show that it is possible to manipulate the information produced by current network analysis to capture a better understanding of some aspects of the system.

7.1.5. Water quality performance in distribution networks

Chapter 5 applies the standard performance assessment framework to the field of water quality in distribution networks. A review of the most important issues regarding the control of water quality is carried out before focusing on the subject of water quality modelling as a prime tool for providing a basis for the performance evaluation programme.

Given the need to make use of one such model, not readily available from other sources, and the possibility of exploring an improved methodology, an accurate multi-parameter dynamic water quality model for physical and chemical parameter simulation was developed and described in detail in the present chapter. The method comprises the simulation of parameter concentration, travel time and source contribution across the network and storage tanks and is illustrated with the help of suitable examples.

It is argued that the model developed, implemented through computer programme PERFORMANCE-Q, is a robust, efficient and, above all, numerically sound method which does not intrinsically introduce any numerical diffusion when modelling the advection and mixing processes. In this respect it is thought to hold clear advantages over some of the documented, established methods.

The method is computationally economical, holding at any one time no more than the actual number of different concentration (or travel time, or source contribution) elements thought to be present in the system. Its flexibility is also important, with various alternative methods of segment aggregation schemes, various transformation functions, the possibility of simulating an array of parameters simultaneously, and travel time and source contribution calculations as well as parameter concentrations.

The performance assessment methodology is followed through the selection of two principal decision variables – parameter concentrations and travel times – for which penalty curves and generalising functions are developed taking into consideration a review of the regulatory aspects in drinking water quality as well as the more diverse operational and technical management policy objectives. Previous water quality examples are completed with the performance evaluation procedure for cases of disinfection improvement and response to contaminant incident, illustrating several of the main properties of the method as applied to water quality.

Having been integrated with the performance assessment methodology, PERFORMANCE-Q constitutes an innovative first proposal for a performance-driven water quality model. The change in philosophy from the traditional approach yields some considerable benefits for the water systems' engineer, not only enabling a direct, performance-oriented analysis that can be easily standardised to afford non-specialists the best informed views of the problem, but also accelerating the process of sensitivity analysis and the gain of knowledge over the system and its behaviour.

7.1.6. Reliability of water distribution networks

Chapter 6 assesses the third and last performance domain, namely the role of reliability evaluation and concepts in water distribution. It begins by reviewing the subject of reliability in water supply and distribution, with the objectives of the performance evaluation framework as much as possible in mind. Direct and indirect methods for assessing reliability are covered, and the latter concentrated upon given their greater suitability to the present analysis.

It is shown that for the purposes of a quantifiable performance assessment method, reliability of a network may be equated to network redundancy, a property for which an entropy-based measure has been developed. Unlike the other two fields of study, it is not possible to define for reliability of water network the same type of clear-cut criteria that allow the performance evaluation framework introduced in Chapter 3 to be fully developed. The above mentioned choice of reliability measure – network flow entropy – eventually adopted in this chapter is arguably the best available methodology for the purpose. It has been formulated, developed and manipulated in this chapter in order to provide the fundamental contribution of a quantifiable and flexible assessment of reliability, and it constitutes the main such proposal currently available.

The applicability of the performance evaluation framework to reliability is then discussed and is found to be less straightforward than in the case of its hydraulic and water quality counterparts. A discussion of the relationships that it is possible to establish between the formulation of the entropy-based function and the variety of meanings that the concept of reliability may have in water supply is carried out. It is found that the expressions available to calculate reliability from a flow-splitting and a flow-joining uncertainty viewpoint help clarify what in reality is at stake when the concept of reliability is formulated against the conflicting objectives of, for example, ensuring continuity of supply and attempting to limit the effects of a pollution incident at source. Reliability is shown to have a much broader meaning than that traditionally employed in water distribution engineering.

The main contributions of the work presented in this chapter are, apart from a comprehensive analysis of reliability measures of performance and the selection of a network flow entropy function as a quantitative measure with potential to integrate the current performance analysis, the development of an alternative, clearly derived formulation that clarifies, complements and validates the established method for the calculation of that function, and the discussion of its properties and use for reliability evaluation in its broader sense.

7.2. SUGGESTIONS FOR FURTHER WORK

7.2.2. Further performance measures

The work developed herein has laid out the foundations for a systematic evaluation of performance issues in water distribution, and has attempted to cover those that, on a first choice, would seem to be the most important. It has by no means, however, exhausted the theme or the applicability of the methodology developed.

First of all, concerning the performance domains covered in this work, it must be said that the measures created could not possibly have been developed to their maximum potential. The nature of the assessment system employed, which is based on highly flexible but also potentially subjective penalty curves, means that only the continuing application of the method in hands-on situations may bring about more definite shapes for those curves and better understanding of their implications.

Secondly, the measures have not been standardised to the full extent. It has not been possible to completely test, for a wide range of situations, whether some aspects are being penalised inconsistently with others, or whether a particular property is influential in more than one curve, in which case coherence should be tested. That would be the case, for example, when defining penalty curves for travel time and for velocity. On that subject, some comments were also made in Chapter 4 about the possible relationship between the velocity measure and the measures involving headloss such as the pressure variation or the available energy ones.

On the other hand, there is certainly much room for defining new measures, both in the areas explored in this work and in other areas as well. One of the main areas for concern in water distribution is the amount of water that is continuously being wasted through leakage, in virtually all distribution systems around the world. Leakage control is a fairly complex topic which has been the subject of a good many studies (see Germanopoulos *et al.*, 1986, Xu, 1990, Farley and Martin, 1994, Gledhill, 1994, among others) and codes of practice (WAA/WRC, 1980, WRC, 1994 and 1994a). Methodologies such as the model presented by Xu (1990) for minimisation of system leakage provide ample scope, for example, for the definition of a simulation-based penalty curve comparing actual levels with optimal levels, or pitting the operational solution against the best possible operational scenarios achieved with optimal valve control. Many other calculations simulating leakage levels through pressure-dependent demands are possible.

As for the continuing refinement of some of the measures introduced in the present study, it is quite clear that the water quality measures proposed necessitate the development of better informed penalty curves than those presented.

7.2.2. Applications of performance assessment methodologies

The performance analysis approach followed in this work attempts to somehow reduce to a quantified, comparable and dimensionless basis a complex analysis problem which is by nature multi-parameter and multi-criterion: the modelling and optimisation of water distribution networks. This is a field where the engineer attempts to make the right decision in the presence of different, often conflicting objectives. It has been mentioned before that traditionally, the problem is circumvented by means of considerable simplification: building and/ or running costs are minimised subject to a simple set of hydraulic constraints. Reliability is introduced chiefly by adding loops and incorporating excess capacity where it seems appropriate, and water quality properties are seldom looked into, other than by avoiding low velocities.

In essence, the approach proposed here could provide a good basis to express those often conflicting objectives in a consistent way, and lead to an ideal formalisation of multi-criterion analysis for water supply. The process, which in many senses is based in a change of variable mechanism, could theoretically be taken to a level of development where all the performance measures, or a set of them chosen for a particular analysis, would be compatible with one another, that is to say, *calibrated* in some way within a global system in order to be able to work together. The challenge that is placed by this perspective is whether it might be developed to a standard that might be adequate for its utilisation in optimisation of operation and design of water networks. No doubt the weighting of the different objectives/ measures would entail considerable difficulty as the number of "degrees of freedom " in the formulation of the problem would increase. On the other hand, the size of the mathematical problem to solve in each iteration could increase to unfeasible levels, especially if heavy additional calculations are needed, such as for the water quality measures which imply the simultaneous use of a water quality simulator. However, a starting point might be found by adopting extremely simple penalty curves, and work from there by increasing the sophistication in small steps.

The concept of a performance-driven (rather than merely cost-driven), or cost driven but performance constrained optimisation, is attractive in the sense that it could lead to efficient design or operation of multi-criterion optimal behaviour solutions. One of the many benefits would be the possibility of finding marginal costs for unit increases in each area of performance. The flexibility of the performance framework does allow for the incorporation of almost any type of sensitivity in many different areas of water network behaviour and properties.

7.2.3. Other areas

Other areas of future development could easily be suggested in the field of water quality modelling, as already mentioned on various occasions throughout Chapter 5. Most of the significant advancements that are still to be made on the type of model described have to do with the lack of sufficiently tested relationships to describe some of the phenomena involved. The development work that is needed in those areas is essentially experimental, but unavoidable if any worthwhile improvements are to be achieved over the existing water quality modelling methods in distribution networks.

The most crucial topic, in terms of the validity of the current models, is definitely the development of efficient water quality transformation models to improve or replace Eqs.5.30 to 5.34, in the formulation presented in Chapter 5. It has been seen how influential the transformation model for non-conservative substances can be in the global modelling process, particularly when taking into consideration the level of accuracy achieved in the advection/dispersion/mixing model. Attention should be paid to the factors that may influence the decay or growth of the various parameters, and attempt to take advantage of the possibility offered by the method described in Chapter 5 for modelling various parameters simultaneously to develop multi-parameter transformation models. In particular, it would be important to take into account the combined effect on the parameter's concentration of such factors as water temperature and pH, as well as other substances or elements present in the water that may influence it. Equally, the consideration of flow velocity, which is generally thought to have a very definite effect on the interaction of many waterborne substances with the pipe walls, ought to be paid more attention to. All these effects can be analysed with the help of the model developed in this work.

Equally, the experimental development of efficient and realistic models for water quality in storage devices is currently identified as very important, being one of the current research priorities of USEPA (Clark, 1993a and 1993b). The effect of storage in the quality of the water that circulates in a supply and distribution network is as significant as it is undocumented.

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APPENDICES

A. NETWORK DATA FOR THE EXAMPLES USED IN THE TEXT

B. A SUMMARY OF GRAPH THEORY TERMINOLOGY AND CONCEPTS

C. CALCULATION OF MAXIMUM ENTROPY FLOWS IN NETWORKS

APPENDIX A

NETWORK DATA FOR THE EXAMPLES USED IN THE TEXT

A.1. NETWORKS FOR THE EXAMPLES IN CHAPTER 4

A.1.1. Town A

System analysis time file

The system analysis time file for Town A was built by taking the average demand scenario as that of the off-peak demand time file T_CARNOP.TIM, which corresponds to the 15:00 snapshot of the 24 hour simulation.

Since it shows very little level variation during the 24 hour simulation, the WITHRES variable head reservoir was kept at the same level throughout, 80.76 m, by specifying a constant level trajectory. However, the WHINN reservoir was modelled by a descending level trajectory, starting at 83.00 m (for 0.25) and dropping all the way to 60.00 m (for 4.00), to replicate the significant variation in the import it models.

The tables below include the topological data, network characteristics and the hydraulic solution for the average demand load.

NCARSYS : Calibrated Network Model - Town A

Snapshot time: 00/01:00C
Selected area: All
Special node selection: Off
Item type: Node

Node Name	Node Type	Area	Total Demand	Ground Level	Total Head	Available Head	Supply
0105		1	0.00	63.4	79.99	16.55	
0110		1	2.15	56.4	78.65	22.29	
01100		3	0.60	18.9	72.62	53.72	
01105		3	3.10	21.0	72.62	51.62	
0111		1	0.00	60.0	78.65	18.65	
01110		1	0.40	14.0	73.75	59.75	
01115		1	0.19	9.6	73.49	63.88	
01120		1	1.72	7.0	73.46	66.46	
01125		1	0.86	25.0	73.01	48.01	
01130		1	0.77	7.2	72.99	65.81	
01135		1	0.00	25.0	73.07	48.07	
01140		1	1.69	23.4	72.83	49.41	
01145		1	1.72	22.0	72.55	50.55	
0115		1	0.00	64.0	78.54	14.54	
01150		3	0.00	22.9	77.10	54.24	
0120		2	0.00	64.0	165.56	101.60	
0125D	PMPD	2	0.00	66.8	165.57	98.81	
0125U	PMPU	1	0.00	65.6	78.53	12.88	
0130		2	0.13	73.0	165.46	92.46	
0135		2	0.25	89.0	164.73	75.73	
0136		2	0.00	103.9	164.73	60.84	
0140		2	4.55	85.5	164.28	78.78	
0145		1	0.01	48.0	75.43	27.43	
0150		1	0.00	28.0	75.15	47.15	
0153		1	0.00	28.0	74.90	46.90	
0154		1	0.00	28.0	78.65	50.65	
0155		3	0.23	28.0	76.45	48.45	
0156		1	0.00	28.0	78.65	50.65	
0157		1	0.00	28.0	74.77	46.77	
0158D	PMPD	1	0.00	23.9	78.65	54.78	
0158U	PMPU	3	0.00	24.5	76.45	51.91	
0159D	NRVD	1	0.00	24.5	78.65	54.11	
0159U	NRVU	3	0.00	24.5	76.45	51.91	
0160		3	1.07	28.0	76.45	48.45	
0161		1	1.40	32.6	74.05	41.46	
0162		1	0.75	28.0	74.02	46.02	
0163		1	1.20	28.0	73.97	45.97	
0164		1	0.00	28.0	74.35	46.35	
0165		3	0.00	27.0	76.45	49.45	
0170		3	0.93	24.9	76.29	51.35	
0175		3	1.63	37.7	76.26	38.56	
0180		3	1.10	20.0	76.28	56.28	
0181		1	0.00	20.0	73.94	53.94	
0185		3	3.73	26.0	74.15	48.15	
0190		3	2.55	24.2	74.01	49.79	
0194		3	0.00	17.0	73.94	56.94	

0195		3	1.12	18.0	72.69	54.69	
WART		1	0.00	44.6	72.83	28.22	
*WHINN	RESR	3	0.00	50.0	77.10	27.10	16.05
*WITHRES	RESR	1	0.00	78.9	80.76	1.88	17.78

NCARSYS : Calibrated Network Model - Town A

Snapshot time: 00/01:00C
Selected area: All
Special node selection: Off
Item type: Pipe

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
0105	0110	217	1428.	2.0000	11.95	0.32	1.34	0.9
0105	0111	166	1428.	2.0000	5.82	0.27	1.34	0.9
0105	WITHRES	166	162.	0.2000	-17.78	-0.82	-0.77	-4.7
0110	0111	217	5.	0.1000	4.88	0.13	0.00	0.1
0110	0115	154	185.	0.0600	4.93	0.26	0.10	0.6
0110	0154	217	1615.	0.1000	0.00	0.00	0.00	0.0
01100	01105	101	305.	0.1000	0.03	0.00	0.00	0.0
01100	0195	102	138.	20.0	-0.64	-0.08	-0.07	-0.5
01105	0195	154	275.	0.0600	-3.06	-0.16	-0.07	-0.2
0111	0145	166	1130.	1.5000	10.70	0.49	3.22	2.8
01110	01115	178	290.	2.0000	6.94	0.28	0.27	0.9
01110	0181	178	180.	2.0000	-7.34	-0.29	-0.18	-1.0
01115	01120	202	385.	0.0600	3.38	0.11	0.03	0.1
01115	01120	202	385.	0.0600	3.38	0.11	0.03	0.1
01115	0180	229	470.	3.5000	0.	0.	0.	0.
01120	01125	102	810.	2.0000	1.20	0.15	0.44	0.5
01120	01135	145	845.	0.0100	3.84	0.23	0.39	0.5
01125	01130	102	520.	2.0000	0.34	0.04	0.02	0.0
01130	01135	145	155.	0.0100	-3.84	-0.23	-0.07	-0.5
01130	01140	154	545.	0.1000	3.41	0.18	0.16	0.3
01140	01145	102	204.	3.5000	1.72	0.21	0.29	1.4
01140	WART	77	340.	0.1000	0.00	0.00	0.00	0.0
0115	0120	154	10.	0.0600	0.	0.	0.	0.
0115	0125U	152	10.	1.5000	4.93	0.27	0.01	1.0
01150	0165	310	2714.	1.5000	16.05	0.21	0.65	0.2
01150	WHINN	310	10.	1.5000	-16.05	-0.21	0.00	-0.2
0120	0125D	152	10.	0.5000	-4.93	-0.27	-0.01	-0.8
0120	0130	154	175.	0.0200	4.93	0.26	0.10	0.5
0130	0135	154	1410.	0.0200	4.80	0.26	0.73	0.5
0135	0136	77	1130.	0.1000	0.00	0.00	0.00	0.0
0135	0140	154	960.	0.0200	4.55	0.24	0.45	0.5
0145	0150	217	395.	1.5000	10.69	0.29	0.27	0.7
0150	0153	166	90.	1.5000	10.69	0.49	0.26	2.8
0153	0154	166	5.	0.1000	0.	0.	0.	0.
0153	0157	166	47.	1.5000	10.69	0.49	0.13	2.8
0154	0156	217	47.	0.1000	0.00	0.00	0.00	0.0
0155	0156	217	10.	0.1000	0.	0.	0.	0.
0155	0157	166	5.	0.1000	0.	0.	0.	0.
0155	0158U	229	10.	3.5000	0.00	0.00	0.00	0.0

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
0155	0160	217	145.	0.1000	-0.23	-0.01	0.00	0.0
0156	0158D	229	10.	3.5000	0.00	0.00	0.00	0.0
0157	0164	166	145.	1.5000	10.69	0.49	0.41	2.8
0158D	0159D	152	5.	3.5000	0.00	0.00	0.00	0.0
0158U	0159U	152	5.	339.0	0.00	0.00	0.00	0.0
0160	0164	178	5.	3.5000	0.	0.	0.	0.
0160	0165	229	233.	1.5000	-1.30	-0.03	0.00	0.0
0161	0162	152	265.	3.5000	1.30	0.07	0.03	0.1
0161	0163	94	460.	0.1000	0.65	0.09	0.08	0.2
0161	0164	152	224.	20.0	-3.35	-0.18	-0.31	-1.4
0162	0163	102	340.	3.5000	0.55	0.07	0.05	0.2
0164	0181	166	520.	0.0600	7.34	0.34	0.42	0.8
0165	0170	217	155.	0.6000	14.75	0.40	0.16	1.0
0170	0175	154	340.	0.1000	1.63	0.09	0.03	0.1
0170	0180	217	140.	0.1000	3.90	0.11	0.01	0.1
0170	0185	152	255.	20.0	8.30	0.46	2.14	8.4
0180	0181	152	5.	3.5000	0.	0.	0.	0.
0180	0185	102	235.	20.0	2.80	0.34	2.13	9.1
0185	0190	154	87.	0.6000	7.37	0.40	0.14	1.6
0190	0194	154	95.	0.6000	4.82	0.26	0.07	0.7
0194	0195	152	440.	20.0	4.82	0.27	1.25	2.8

NCARSYS : Calibrated Network Model - Town A

Snapshot time: 00/01:00C
Selected Area: All
Special node selection: Off
Item type: TSP

Inlet Node	Outlet Node	Pump Type	Flow	Lift	Status
0125U	0125D	Boos	4.93	87.03	On
0158U	0158D	Boos			Off

NCARSYS : Calibrated Network Model - Town A

Snapshot time: 00/01:00C
Selected Area: All
Special node selection: Off
Item type: Valve

Inlet Node	Outlet Node	Valve Type	Setting	Inlet Head	Outlet Head	Flow	Status	Control Node
0159U	0159D	NRV		76.45	78.65		Shut	

NCARSYS : Calibrated Network Model - Town A

Snapshot time: 00/01:00C
Selected Area: All
Special node selection: Off
Item type: Var Resvr

Node Name	Bottom Water Level	Top Water Level	Actual Water Level
*WHINN	50.00	100.00	77.1000
*WITHRES	77.88	81.08	80.7600

A.1.2. Town B

System analysis time file

The system analysis time file for Town B was built by taking the 16:00 snapshot in the 24 hour simulation as the average demand scenario to be factored. The operation of this system is less straightforward than the other two and so this simulation translates only one of its possible configurations, albeit certainly the one that is used during most of the daytime peak hours.

The tables below include the topological data, network characteristics and the hydraulic solution for the average demand load.

```
ALS01STC: Town B Base Network

Snapshot time: 01/01:00C
Selected area: All
Special node selection: Off
Item type: Node
```

Node Name	Node Type	Area	Total Demand	Ground Level	Total Head	Available Head	Supply
AF01	PMPD	1	0.00	60.0	153.13	93.13	
BA01	NRVD	1	0.00	85.0	87.60	2.60	
BA02	NRVU	1	0.00	85.0	87.57	2.57	
BA03	NRVU	1	0.00	85.0	87.58	2.58	
BA04	NRVD	1	0.00	85.0	87.58	2.58	
BA05		1	0.00	85.0	87.57	2.57	
BA06		1	0.00	85.0	87.56	2.56	
BA07		1	0.00	85.0	87.56	2.56	
BA08		1	0.00	85.0	162.23	77.23	
BA09	PMPU	1	0.00	85.0	87.44	2.44	
BA10	PMPD	1	0.00	85.0	162.41	77.41	
BA11	PMPU	1	0.00	85.0	87.44	2.44	
BA12	PMPD	1	0.00	85.0	162.41	77.41	
BA13	PMPU	1	0.00	85.0	87.44	2.44	
BA14	PMPD	1	0.00	85.0	162.41	77.41	
BA15		1	0.00	85.0	162.41	77.41	
BA16		1	0.00	85.0	162.23	77.23	
BA17		1	0.00	85.0	87.45	2.45	
BE01		1	0.00	68.0	140.66	72.66	
C-GREEN	RESR	1	0.00	60.5	90.00	29.52	26.90
CRW		15	54.50	84.0	87.56	3.56	

Node Name	Node Type	Area	Total Demand	Ground Level	Total Head	Available Head	Supply
DC01		1	0.00	85.0	147.43	62.43	
DC02		1	0.05	85.0	147.43	62.43	
DE01		1	0.00	85.0	133.91	48.91	
DE02		1	0.22	90.0	133.89	43.89	
EE01		1	0.02	90.0	133.61	43.61	
EE02		1	0.00	78.4	133.59	55.18	
EE03		1	0.00	84.0	133.08	49.08	
EE04		1	0.00	81.4	131.66	50.23	
EE05		1	0.00	85.0	131.16	46.16	
EF01		1	0.00	84.0	122.30	38.30	
EF02		1	2.93	82.2	122.28	40.09	
EF03		1	0.00	83.0	123.10	40.10	
EF04		1	0.00	84.0	123.67	39.67	
EF05		1	0.00	84.0	123.43	39.43	
EN01		4	0.00	74.0	118.26	44.26	
EN02		4	0.00	74.0	118.26	44.26	
EO01		4	0.80	72.4	118.24	45.79	
FE01		1	1.47	84.5	123.88	39.36	
FE02		1	0.00	85.0	122.52	37.52	
FF01		1	0.00	87.0	129.44	42.44	
FF02		1	0.00	87.0	129.09	42.09	
FF03		1	2.57	87.0	122.22	35.22	
FF04		1	0.00	86.0	122.22	36.22	
FF05		1	2.77	82.8	122.23	39.47	
FF06		1	0.00	85.0	124.59	39.59	
FF07		1	2.50	86.0	124.82	38.82	
FF08		1	0.00	85.4	125.05	39.68	
FF09		1	0.00	87.0	122.29	35.29	
FF10		1	0.00	87.0	122.99	35.99	
FF11		1	0.00	88.2	126.99	38.79	
FF12		1	0.00	88.0	126.38	38.38	
FF13		1	0.72	88.0	126.27	38.27	
FF14		1	0.00	89.0	123.93	34.93	
FF15		1	1.77	88.0	123.80	35.80	
FG01		1	0.00	84.0	124.29	40.29	
FG02		1	0.00	84.0	124.25	40.25	
FG03		1	3.18	84.0	123.97	39.97	
FG04		1	0.00	86.0	124.25	38.25	
FG05		1	2.81	85.0	123.96	38.96	
FG06		1	0.00	85.1	123.97	38.90	
FI01		2	0.00	80.0	110.56	30.56	
FI02		2	1.49	77.0	110.51	33.52	
FJ01		2	0.00	78.0	110.56	32.56	
FM01		4	0.43	73.0	118.33	45.33	
FM02		4	0.00	74.4	118.33	43.95	
FO01		4	0.00	75.1	118.22	43.17	
FO02		4	0.00	76.0	118.22	42.22	
FPACK		10	7.20	92.0	110.89	18.89	
GE01		1	0.01	106.0	125.27	19.27	
GF01		1	1.20	89.0	125.08	36.08	
GF02		1	0.00	89.0	125.68	36.68	
GF03		1	2.61	84.0	125.65	41.65	
GF04		1	0.00	89.6	125.80	36.18	

Node Name	Node Type	Area	Total Demand	Ground Level	Total Head	Available Head	Supply
GF05		1	1.02	87.0	124.94	37.94	
GF06		1	0.00	87.0	125.93	38.93	
GF07		1	0.00	85.0	125.39	40.39	
GF08		1	1.14	87.0	125.30	38.30	
GF09		1	0.00	87.0	125.30	38.30	
GF10		1	1.27	88.0	124.98	36.98	
GF11		1	0.00	89.0	124.90	35.90	
GF12		1	0.00	90.0	124.52	34.52	
GF13		1	0.60	90.0	124.59	34.59	
GF14		1	1.22	92.0	124.29	32.29	
GF15		1	0.00	89.0	124.31	35.35	
GF16		1	4.62	89.1	123.28	34.14	
GF17		1	2.33	93.0	119.71	26.71	
GF18		1	0.00	88.0	119.73	31.73	
GF19		1	0.00	88.0	119.95	31.95	
GG01		1	0.00	88.0	124.01	36.01	
GG02		1	1.37	80.0	123.47	43.47	
GG03		1	2.93	80.0	123.31	43.31	
GG04		1	0.00	90.1	123.06	32.93	
GG05		1	1.19	89.1	123.03	33.92	
GI01		2	0.23	83.5	116.30	32.84	
GJ01		2	0.00	81.7	118.27	36.57	
GL01		4	0.00	71.0	119.28	48.28	
GL02		4	0.23	74.7	119.28	44.56	
GN01		4	0.33	72.9	118.19	45.27	
GORSTY1I	RESR	1	0.00	86.0	87.60	1.60	0.00
*GORSTY1O	RESR	1	0.00	86.0	87.60	1.60	159.73
HE01		1	1.98	93.4	125.05	31.68	
HE02		1	0.00	92.6	125.05	32.49	
HE03		1	0.00	96.0	125.34	29.34	
HE04		1	0.00	96.0	125.12	29.12	
HE05		1	0.00	95.0	125.23	30.23	
HE06		1	0.00	96.9	125.11	28.24	
HE07		1	3.89	97.0	125.08	28.08	
HE08		1	0.00	105.6	125.40	19.77	
HE09		1	0.00	98.0	125.39	27.39	
HE10		1	1.96	98.0	125.38	27.38	
HF01		1	0.00	93.0	125.04	32.04	
HF02		1	0.00	93.0	125.42	32.42	
HF03		1	4.33	94.0	125.35	31.35	
HF04		1	0.00	94.0	125.34	31.34	
HF05		1	0.00	96.0	125.14	29.14	
HF06		1	0.00	97.0	125.14	28.14	
HF07		1	0.00	98.0	125.22	27.22	
HF08		1	0.00	98.0	125.29	27.29	
HF09		1	1.84	98.0	125.38	27.38	
HF10		1	0.00	98.9	125.38	26.44	
HF11		1	2.91	90.0	119.48	29.48	
HF12		1	2.71	90.0	119.47	29.47	
HF13		1	0.93	91.5	119.48	27.99	
HF14		1	1.29	98.0	119.73	21.73	
HF15		1	0.00	92.0	119.60	27.60	
HF16		1	0.00	89.0	123.78	34.78	

Node Name	Node Type	Area	Total Demand	Ground Level	Total Head	Available Head	Supply
HF17		1	0.00	89.0	123.73	34.73	
HF18		1	1.34	89.0	123.73	34.73	
HF19		1	1.51	92.0	110.90	18.90	
HG01		1	0.00	92.8	123.42	30.63	
HG02		1	0.98	91.0	122.87	31.87	
HG03		1	0.00	91.0	122.91	31.91	
HG04		1	0.14	90.0	122.60	32.60	
HG05		1	0.00	91.0	122.22	31.22	
HG06		1	0.00	91.0	122.20	31.20	
HG07		1	0.00	92.0	123.41	31.41	
HG08		1	1.47	92.0	123.40	31.40	
HG09		1	0.00	92.4	121.96	29.60	
HG10		1	0.00	91.0	121.94	30.92	
HH01		3	0.00	91.7	121.82	30.09	
HH02		16	15.45	92.3	121.69	29.39	
HH03		3	0.00	92.0	121.61	29.61	
HH04		3	0.00	93.0	119.87	26.87	
HH05		3	6.45	92.7	118.45	25.79	
HH06		3	7.31	92.1	117.87	25.79	
HH07		3	0.00	93.0	117.89	24.89	
HJ01		2	0.47	85.4	121.05	35.67	
HOSPITAL		13	4.06	73.0	118.33	45.33	
HURL		1	0.00	84.0	87.56	3.56	
IE01	NRVU	1	0.00	102.0	125.45	23.45	
IE02	NRVD	1	0.00	102.0	125.45	23.45	
IE03	NRVU	1	0.00	102.0	125.43	23.43	
IE04	NRVD	1	0.00	102.0	125.47	23.47	
IF01		1	0.77	92.0	123.33	31.33	
IF02		1	0.67	92.0	123.58	31.58	
IG01		1	0.00	100.9	123.47	22.58	
IG02		1	2.72	90.0	123.42	33.42	
JARROBS		11	12.57	92.0	110.86	18.86	
LEAK		17	15.87	89.0	110.53	21.53	
LININ	RESR	1	0.00	123.0	125.47	2.47	0.00
*LINLEY	RESR	1	0.00	123.0	125.47	2.47	34.14
ROF		14	18.60	90.0	133.52	43.52	
TWYFORDS		12	0.81	92.0	119.60	27.60	

ALS01STC: Town B Base Network

Snapshot time: 01/01:00C
Selected area: All
Special node selection: Off
Item type: Pipe

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
AF01	BE01	200	1510.	3.0000	26.90	0.86	12.48	8.3
BA01	GORSTY1I	450	40.	0.0300	0.00	0.00	0.00	0.0
BA02	BA05	450	20.	0.0300	0.00	0.00	0.00	0.0
BA03	GORSTY1O	600	45.	0.0300	-159.73	-0.56	-0.02	-0.4

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
BA04	BA05	600	30.	0.0300	159.73	0.56	0.01	0.4
BA05	BA06	600	10.	0.0300	159.73	0.56	0.00	0.4
BA06	BA07	600	10.	0.0300	159.73	0.56	0.00	0.4
BA06	HURL	600	20.	0.0300	0.00	0.00	0.00	0.0
BA07	BA17	300	20.	0.0300	105.23	1.49	0.11	5.7
BA07	CRW	600	35.	0.0300	54.50	0.19	0.00	0.1
BA08	BA16	75	10.	10.0	0.00	0.00	0.00	0.0
BA09	BA17	300	5.	0.0300	-35.08	-0.50	0.00	-0.8
BA10	BA15	300	5.	0.0300	35.08	0.50	0.00	0.8
BA11	BA17	300	5.	0.0300	-35.08	-0.50	0.00	-0.8
BA12	BA15	300	5.	0.0300	35.08	0.50	0.00	0.8
BA13	BA17	300	5.	0.0300	-35.08	-0.50	0.00	-0.8
BA14	BA15	300	5.	0.0300	35.08	0.50	0.00	0.8
BA15	BA16	300	30.	0.0300	105.23	1.49	0.17	5.7
BA16	DC01	300	2590.	0.0300	105.23	1.49	14.80	5.7
BE01	DE01	150	3220.	10.0	4.81	0.27	6.75	2.1
BE01	DE01	200	2900.	0.0300	22.09	0.70	6.75	2.3
DC01	DC02	75	100.	2.0000	0.05	0.01	0.00	0.0
DC01	EE01	300	2420.	0.0300	105.18	1.49	13.82	5.7
DE01	DE02	75	150.	0.0300	0.22	0.05	0.01	0.1
DE01	EE02	300	700.	0.0300	26.68	0.38	0.32	0.5
EE01	EE02	150	350.	0.0300	1.56	0.09	0.03	0.1
EE01	EE02	200	350.	0.0300	3.41	0.11	0.03	0.1
EE01	EE04	300	550.	0.0300	81.60	1.15	1.96	3.6
EE01	ROF	150	10.	0.3000	18.60	1.05	0.09	9.4
EE02	EE03	100	100.	2.0000	3.50	0.45	0.51	5.1
EE02	EE04	225	400.	3.0000	28.15	0.71	1.93	4.8
EE03	FE01	100	580.	20.0	3.50	0.45	9.19	15.8
EE04	EE05	300	80.	0.0300	109.75	1.55	0.49	6.2
EE05	FF01	300	470.	0.0300	82.81	1.17	1.72	3.7
EE05	FF02	225	470.	3.0000	26.94	0.68	2.08	4.4
EF01	EF02	100	220.	0.3000	0.48	0.06	0.02	0.1
EF01	FE02	75	470.	2.0000	-0.48	-0.11	-0.23	-0.5
EF02	EF03	100	350.	0.3000	-3.07	-0.39	-0.82	-2.4
EF02	FF05	100	380.	0.3000	0.61	0.08	0.05	0.1
EF03	EF05	100	140.	0.3000	-3.07	-0.39	-0.33	-2.4
EF04	EF05	75	65.	10.0	0.92	0.21	0.24	3.6
EF04	FF07	75	320.	10.0	-0.92	-0.21	-1.16	-3.6
EF05	FF15	150	870.	10.0	-2.14	-0.12	-0.37	-0.4
EN01	EN02	175	100.	10.0	0.98	0.04	0.00	0.0
EN01	FM01	175	1810.	10.0	-0.98	-0.04	-0.07	0.0
EN02	EO01	175	430.	10.0	0.98	0.04	0.02	0.0
EO01	FO01	100	670.	10.0	0.18	0.02	0.02	0.0
FE01	FE02	75	90.	20.0	1.50	0.34	1.36	15.1
FE01	FF10	75	460.	20.0	0.53	0.12	0.89	1.9
FE02	FF03	100	330.	10.0	1.02	0.13	0.30	0.9
FF01	FF02	225	10.	10.0	60.65	1.53	0.36	35.8
FF01	FF11	225	510.	10.0	22.16	0.56	2.45	4.8
FF02	FF11	300	515.	0.0300	87.59	1.24	2.09	4.1
FF03	FF04	150	140.	0.0300	-0.39	-0.02	0.00	0.0
FF03	FF09	100	220.	0.0300	-1.15	-0.15	-0.07	-0.3
FF04	FF05	100	160.	0.0300	-0.39	-0.05	-0.01	0.0
FF05	FF06	50	60.	0.0300	-2.55	-1.30	-2.36	-39.4

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
FF06	FF07	100	170.	0.0300	-2.55	-0.32	-0.23	-1.4
FF07	FF08	150	260.	0.0300	-5.97	-0.34	-0.23	-0.9
FF08	FF12	150	360.	0.0300	-13.21	-0.75	-1.33	-3.7
FF08	FG01	150	610.	0.0300	7.24	0.41	0.76	1.2
FF09	FF10	75	80.	20.0	-1.15	-0.26	-0.70	-8.7
FF10	FF14	75	380.	20.0	-0.61	-0.14	-0.94	-2.5
FF11	FF12	150	150.	0.0300	13.93	0.79	0.61	4.1
FF11	GF05	225	390.	3.0000	29.45	0.74	2.06	5.3
FF11	GF06	300	440.	0.0300	66.36	0.94	1.07	2.4
FF12	FF13	75	50.	10.0	0.72	0.16	0.11	2.2
FF14	FF15	75	360.	10.0	0.29	0.07	0.13	0.4
FF14	GF01	75	210.	20.0	-0.91	-0.20	-1.15	-5.5
FF15	FG01	150	620.	10.0	-2.96	-0.17	-0.50	-0.8
FF15	GF05	75	610.	10.0	-0.66	-0.15	-1.14	-1.9
FG01	FG02	150	120.	0.0300	3.72	0.21	0.05	0.4
FG01	FG06	75	240.	10.0	0.56	0.13	0.33	1.4
FG02	FG03	150	750.	0.0300	3.72	0.21	0.28	0.4
FG02	FG04	125	390.	10.0	0.00	0.00	0.00	0.0
FG03	FG05	150	150.	0.0300	0.54	0.03	0.00	0.0
FG05	FG06	150	330.	0.0300	-0.56	-0.03	0.00	0.0
FG05	GG01	150	440.	0.0300	-1.71	-0.10	-0.04	-0.1
FI01	FI02	150	650.	0.0300	1.49	0.08	0.05	0.1
FI01	FJ01	100	110.	10.0	0.00	0.00	0.00	0.0
FI01	GI01	150	940.	0.0300	-17.36	-0.98	-5.74	-6.1
FI01	LEAK	150	2.	3.0000	15.87	0.90	0.03	13.5
FM01	FM02	100	150.	20.0	0.16	0.02	0.01	0.0
FM01	GL01	175	540.	20.0	-5.63	-0.23	-0.95	-1.8
FM01	HOSPITAL	150	10.	0.3000	4.06	0.23	0.01	0.5
FM02	GN01	75	1250.	10.0	0.16	0.04	0.14	0.1
FO01	FO02	100	80.	10.0	0.18	0.02	0.00	0.0
FO02	GN01	100	1010.	10.0	0.18	0.02	0.03	0.0
FPACK	HF19	150	10.	0.3000	-7.20	-0.41	-0.02	-1.5
GE01	GF08	150	470.	0.0300	-1.45	-0.08	-0.03	-0.1
GE01	HE01	150	2015.	2.0000	1.45	0.08	0.22	0.1
GF01	GF02	75	70.	2.0000	-2.10	-0.48	-0.60	-8.6
GF02	GF03	150	90.	2.0000	2.61	0.15	0.03	0.3
GF02	GF04	150	110.	2.0000	-4.72	-0.27	-0.12	-1.1
GF04	GF06	450	190.	3.0000	-66.36	-0.42	-0.13	-0.7
GF04	GF07	125	130.	2.0000	5.01	0.41	0.41	3.1
GF04	GF13	75	300.	2.0000	1.43	0.32	1.21	4.0
GF04	HF02	450	920.	2.0000	55.21	0.35	0.38	0.4
GF05	GF15	225	315.	0.0300	27.77	0.70	0.63	2.0
GF07	GF08	125	190.	0.0300	2.60	0.21	0.09	0.5
GF07	GF10	100	340.	0.0300	2.41	0.31	0.42	1.2
GF08	GF09	150	135.	0.0300	0.00	0.00	0.00	0.0
GF10	GF11	100	230.	0.0300	1.14	0.15	0.08	0.3
GF11	GF12	100	90.	2.0000	3.19	0.41	0.38	4.2
GF11	GF14	225	285.	2.0000	19.95	0.50	0.61	2.1
GF11	HF02	225	200.	2.0000	-21.99	-0.55	-0.52	-2.6
GF12	HF01	75	135.	2.0000	-1.40	-0.32	-0.52	-3.9
GF12	HF19	75	335.	2.0000	4.59	1.04	13.62	40.6
GF13	GF14	75	210.	2.0000	0.83	0.19	0.29	1.4
GF14	GF15	150	685.	0.0300	-0.77	-0.04	-0.02	0.0

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
GF14	GF17	150	240.	2.0000	20.33	1.15	4.58	19.1
GF15	GG01	225	160.	0.0300	27.00	0.68	0.30	1.9
GF16	GF19	100	430.	0.3000	5.72	0.73	3.33	7.7
GF16	GG01	150	410.	0.0300	-8.81	-0.50	-0.73	-1.8
GF16	GG03	150	390.	0.0300	-1.53	-0.09	-0.03	-0.1
GF17	GF18	100	185.	0.3000	-0.49	-0.06	-0.02	-0.1
GF17	HF14	100	290.	0.0300	-0.34	-0.04	-0.01	0.0
GF17	HF15	150	725.	0.3000	2.14	0.12	0.12	0.2
GF17	HF19	100	210.	0.0300	16.69	2.12	8.81	42.0
GF18	GF19	100	135.	0.3000	-2.55	-0.32	-0.22	-1.7
GF18	HF12	100	230.	0.3000	2.06	0.26	0.26	1.1
GF19	HF11	100	190.	0.3000	3.16	0.40	0.47	2.5
GG01	GG02	225	320.	3.0000	16.48	0.41	0.53	1.7
GG02	GG03	150	310.	0.0300	4.46	0.25	0.16	0.5
GG02	GG04	225	580.	3.0000	10.65	0.27	0.41	0.7
GG04	GG05	100	100.	0.0300	1.19	0.15	0.04	0.4
GG04	HG04	225	840.	3.0000	9.46	0.24	0.47	0.6
GI01	GJ01	175	690.	6.0000	-9.63	-0.40	-1.97	-2.9
GI01	HH07	200	1230.	12.0	-7.96	-0.25	-1.59	-1.3
GJ01	HJ01	175	970.	6.0000	-9.63	-0.40	-2.78	-2.9
GL01	GL02	100	110.	2.0000	0.23	0.03	0.00	0.0
GL01	HJ01	175	2440.	2.0000	-5.86	-0.24	-1.76	-0.7
HE01	HE02	150	30.	2.0000	-0.98	-0.06	0.00	-0.1
HE01	HF01	150	430.	2.0000	0.45	0.03	0.01	0.0
HE02	HE04	100	160.	2.0000	-0.98	-0.12	-0.07	-0.4
HE03	HF04	225	220.	2.0000	0.00	0.00	0.00	0.0
HE04	HE05	100	140.	2.0000	-1.37	-0.17	-0.11	-0.8
HE04	HE06	100	150.	2.0000	0.40	0.05	0.01	0.1
HE05	HF03	100	140.	2.0000	-1.37	-0.17	-0.11	-0.8
HE06	HE07	100	50.	2.0000	1.13	0.14	0.03	0.6
HE06	HF07	100	460.	2.0000	-0.74	-0.09	-0.11	-0.2
HE07	HF05	100	70.	2.0000	-1.48	-0.19	-0.07	-0.9
HE07	HF06	100	90.	2.0000	-1.27	-0.16	-0.06	-0.7
HE08	HE09	450	55.	0.0300	34.14	0.21	0.01	0.1
HE08	IE03	450	290.	0.0300	-34.14	-0.21	-0.03	-0.1
HE09	HE10	100	245.	2.0000	0.34	0.04	0.01	0.1
HE09	HF10	450	150.	0.0300	33.80	0.21	0.01	0.1
HE10	HF09	450	310.	0.0300	-1.62	-0.01	0.00	0.0
HF01	HF03	75	165.	2.0000	-0.95	-0.22	-0.30	-1.8
HF02	HF03	225	165.	2.0000	8.94	0.22	0.07	0.4
HF02	HF09	450	740.	0.0300	24.28	0.15	0.04	0.1
HF03	HF04	225	65.	2.0000	1.72	0.04	0.00	0.0
HF03	HF08	100	410.	2.0000	0.56	0.07	0.06	0.1
HF04	HF05	100	160.	2.0000	1.72	0.22	0.20	1.3
HF05	HF06	100	70.	2.0000	0.24	0.03	0.00	0.0
HF06	HF07	100	165.	2.0000	-1.03	-0.13	-0.08	-0.5
HF07	HF08	100	50.	2.0000	-1.77	-0.23	-0.07	-1.3
HF08	HF09	100	145.	2.0000	-1.21	-0.15	-0.09	-0.6
HF09	HF10	450	30.	0.0300	19.61	0.12	0.00	0.0
HF10	HF16	300	980.	0.0300	53.40	0.76	1.59	1.6
HF11	HF12	100	100.	0.3000	0.39	0.05	0.01	0.1
HF11	HF13	100	340.	0.3000	-0.14	-0.02	0.00	0.0
HF12	HF13	100	330.	0.3000	-0.26	-0.03	-0.01	0.0

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
HF13	HF15	100	235.	0.3000	-1.33	-0.17	-0.12	-0.5
HF14	IF01	75	1170.	0.3000	-1.63	-0.37	-3.60	-3.1
HF15	TWYFORDS	150	10.	0.3000	0.81	0.05	0.00	0.0
HF16	HF17	150	60.	0.0300	5.98	0.34	0.05	0.9
HF16	HG01	300	280.	0.0300	47.43	0.67	0.37	1.3
HF17	HF18	150	60.	2.0000	1.34	0.08	0.01	0.1
HF17	IF02	150	145.	2.0000	4.64	0.26	0.15	1.0
HF19	JARROBS	150	10.	0.3000	12.57	0.71	0.04	4.4
HG01	HG03	150	470.	0.0300	6.72	0.38	0.51	1.1
HG01	HG07	100	255.	2.0000	0.22	0.03	0.01	0.0
HG01	HG09	300	870.	2.0000	38.08	0.54	1.46	1.7
HG01	IF01	150	300.	2.0000	2.41	0.14	0.09	0.3
HG02	HG03	75	40.	0.0300	-0.98	-0.22	-0.04	-1.0
HG03	HG04	150	380.	0.0300	5.74	0.32	0.31	0.8
HG04	HG05	225	310.	2.0000	15.06	0.38	0.38	1.2
HG05	HG06	225	10.	2.0000	15.06	0.38	0.01	1.2
HG06	HG10	225	215.	2.0000	15.06	0.38	0.26	1.2
HG07	HG08	100	95.	2.0000	0.51	0.07	0.01	0.1
HG07	IG02	100	385.	0.0300	-0.29	-0.04	-0.01	0.0
HG08	IG01	100	300.	0.0300	-0.96	-0.12	-0.07	-0.2
HG09	HG10	300	30.	2.0000	22.12	0.31	0.02	0.6
HG09	HJ01	300	3010.	2.0000	15.96	0.23	0.91	0.3
HG10	HH01	300	65.	3.0000	37.17	0.53	0.12	1.8
HH01	HH02	200	55.	2.0000	15.45	0.49	0.13	2.4
HH01	HH03	200	45.	2.0000	21.72	0.69	0.21	4.7
HH03	HH04	200	370.	2.0000	21.72	0.69	1.74	4.7
HH04	HH05	150	210.	2.0000	12.02	0.68	1.41	6.7
HH04	HH07	150	450.	2.0000	9.70	0.55	1.98	4.4
HH05	HH06	150	400.	2.0000	5.57	0.32	0.59	1.5
HH06	HH07	150	140.	2.0000	-1.74	-0.10	-0.02	-0.2
IE01	LINLEY	450	250.	0.0300	-34.14	-0.21	-0.03	-0.1
IE02	IE03	450	190.	0.0300	34.14	0.21	0.02	0.1
IE04	LININ	450	80.	0.0300	0.00	0.00	0.00	0.0
IF02	IG01	150	145.	2.0000	3.97	0.22	0.11	0.8
IG01	IG02	150	110.	2.0000	3.01	0.17	0.05	0.4

ALS01STC: Town B Base Network

Snapshot time: 01/01:00C
Selected Area: All
Special node selection: Off
Item type: TSP

Inlet Node	Outlet Node	Pump Type	Flow	Lift	Status
BA09	BA10	Boos	35.08	74.97	On
BA11	BA12	Boos	35.08	74.97	On
BA13	BA14	Boos	35.08	74.97	On
C-GREEN	AF01	Srce	26.90	63.13	On

ALS01STC: Town B Base Network

Snapshot time: 01/01:00C
Selected Area: All
Special node selection: Off
Item type: Valve

Inlet Node	Outlet Node	Valve Type	Setting	Inlet Head	Outlet Head	Flow	Status	Control Node
BA02	BA01	NRV		87.57	87.60		Shut	
IE03	IE04	NRV		125.43	125.47		Shut	
IE01	IE02	NRV		125.45	125.45	34.14	Open	
BA03	BA04	NRV		87.58	87.58	159.73	Open	

ALS01STC: Town B Base Network

Snapshot time: 01/01:00C
Selected Area: All
Special node selection: Off
Item type: Var Resvr

Node Name	Bottom Water Level	Top Water Level	Actual Water Level
*GORSTY10	84.60	91.50	87.6000
*LINLEY	122.60	126.62	125.4710

A.1.3. Town C

System analysis time file

The system analysis time file for Town C was built by taking the average demand scenario to be that of the 16:00 snapshot in the 24 hour simulation. The way the original demand factors were specified in this model simplified the procedure, because an average demand could simply be defined as a demand factor of 1.

The network is supplied from a single reservoir, and the original model included an artificial negative demand node to simulate a variation in the incoming supply level. This is no longer necessary as reservoir trajectories can now be specified in Watnet 4, hence nodes 1I and 1S and demand area 13 were disposed of. For the system analysis simulation, a trajectory was defined starting at 43.55 m and ending at 42 m, in order to simulate the decrease in supply level with the load factor.

The complexity of the demand distribution in the network of Town C makes it difficult to produce a system analysis time file without real insight into the consumption characteristics of some of the large consumers. This is also true of the seasonal variations that may affect some of the demand areas due to the influx of tourists, for example. The file produced here, which amplifies demand types 1 to 4 in all demand areas equally, is but one possible configuration.

The tables below include the topological data, network characteristics and the hydraulic solution for the average demand load.

NLYMSYS : TOWN C NETWORK

Snapshot time: 00/01:00C
Selected area: All
Special node selection: Off
Item type: Node

Node Name	Node Type	Area	Total Demand	Ground Level	Total Head	Available Head	Supply
102		5	3.47	12.4	39.70	27.27	
114		2	1.02	7.3	39.57	32.27	
117		2	0.56	7.3	39.56	32.26	
117A		4	3.36	7.3	39.56	32.26	
12		3	0.82	7.0	40.41	33.41	
120		2	2.06	9.1	39.42	30.32	
123		2	0.22	7.2	39.55	32.35	
126		2	0.13	5.8	39.54	33.74	
127		2	0.12	8.0	39.49	31.46	
128		2	0.55	4.5	39.47	34.93	
129		1	0.00	30.8	42.97	12.17	
132		2	0.70	27.4	42.30	14.90	
135		2	0.35	24.4	41.60	17.20	
138		2	0.59	25.6	41.49	15.89	
141		2	0.08	27.4	41.44	14.04	
144		2	0.18	28.0	41.44	13.44	
145		2	0.77	10.9	41.30	30.44	
146		2	0.55	7.6	41.29	33.69	
147		2	0.29	26.5	41.18	14.66	
15		3	0.62	8.8	40.27	31.47	
150		2	1.52	22.5	39.94	17.44	
153		2	0.53	14.3	39.80	25.50	
154		2	1.11	14.3	39.77	25.47	
156		1	0.00	26.0	41.81	15.81	
159		1	0.00	23.4	39.96	16.58	
162		1	0.02	4.6	39.04	34.44	
165		1	1.33	6.3	38.13	31.84	
166		1	0.00	4.3	38.06	33.76	
168		1	0.52	4.3	37.33	33.03	
171		1	1.13	3.8	36.75	32.98	
174		1	3.04	4.5	35.29	30.76	
176		1	1.41	7.3	35.01	27.71	
177		1	0.00	4.6	33.55	28.95	
180		1	0.99	3.7	33.55	29.82	
183		1	4.67	7.0	33.61	26.61	
186		1	1.48	6.4	34.22	27.82	
189		1	1.02	4.7	34.58	29.88	
192		1	1.68	5.8	34.72	28.92	
198		1	0.61	6.4	34.92	28.52	
*1R	RESR	1	0.00	38.1	43.16	5.06	444.25
2		1	0.00	30.4	43.16	12.72	
204		1	2.06	3.4	36.59	33.19	
207		1	0.62	3.6	36.70	33.10	
21		3	1.02	15.0	40.28	25.28	
210		1	1.08	5.2	34.58	29.38	
210A		8	2.36	5.2	34.58	29.38	

Node Name	Node Type	Area	Total Demand	Ground Level	Total Head	Available Head	Supply
213		1	1.29	5.8	34.72	28.92	
215		1	1.34	7.0	34.92	27.92	
216		1	1.75	7.3	34.97	27.67	
217		1	0.42	7.2	35.01	27.80	
218		1	0.09	7.3	35.01	27.71	
219		1	1.91	7.6	34.99	27.39	
222		1	1.49	7.0	34.54	27.54	
225		1	0.00	7.0	34.58	27.58	
226		1	1.08	11.3	34.58	23.28	
228		1	1.48	11.3	34.58	23.28	
229		1	0.43	11.0	34.59	23.59	
231		1	1.03	6.7	34.61	27.94	
233		1	0.18	5.0	38.02	33.02	
234		1	0.15	6.4	36.81	30.42	
237		1	0.94	6.4	36.44	30.04	
24		3	0.45	19.2	40.51	21.31	
240		1	0.08	6.7	36.24	29.54	
243		1	0.70	7.3	36.17	28.87	
246		1	0.55	9.0	35.80	26.84	
249		1	0.63	8.8	35.62	26.82	
252		1	0.54	7.3	36.33	29.03	
255		1	0.54	7.3	36.10	28.80	
258		1	0.89	4.6	36.08	31.48	
259		1	0.00	4.6	36.08	31.48	
261		1	2.13	7.3	36.05	28.75	
263		1	0.00	7.3	35.96	28.66	
264		1	0.84	7.3	35.40	28.10	
267		1	1.77	7.3	35.19	27.89	
27		3	1.08	19.0	40.52	21.52	
270		1	1.49	7.6	35.36	27.76	
271		1	0.00	7.6	35.23	27.63	
273		1	0.77	8.2	35.20	27.00	
276		1	1.86	8.5	35.19	26.69	
279		1	1.25	7.6	34.70	27.10	
282		1	0.62	8.8	34.99	26.19	
284		1	0.18	9.0	35.55	26.55	
285		1	0.45	8.8	35.55	26.75	
288		1	0.68	8.5	35.36	26.86	
289		1	0.00	8.4	35.20	26.78	
291		1	1.45	8.8	35.19	26.39	
293		1	0.11	9.7	35.12	25.43	
294		1	1.08	8.2	35.00	26.79	
295		1	0.36	8.5	35.30	26.80	
296		1	0.00	7.6	35.28	27.68	
297		1	0.53	7.6	35.26	27.66	
298		1	0.00	7.6	35.22	27.62	
300		1	0.00	8.8	35.21	26.41	
303		1	0.00	13.6	40.72	27.09	
304		1	0.00	13.6	40.39	26.76	
306		1	0.00	10.7	38.80	28.10	
309		1	0.02	4.9	38.32	33.42	
311		1	0.00	7.6	37.64	30.08	
312		1	0.55	7.6	36.76	29.20	

Node Name	Node Type	Area	Total Demand	Ground Level	Total Head	Available Head	Supply
315		1	0.74	8.2	36.57	28.37	
318		1	0.20	7.6	36.49	28.89	
321		1	0.90	8.5	36.37	27.87	
324		1	0.40	8.5	36.28	27.78	
327		1	0.13	8.3	36.20	27.92	
330		1	0.81	8.5	36.16	27.66	
333		1	1.41	8.8	36.12	27.32	
336		1	0.21	10.0	36.10	26.10	
339		1	0.96	6.8	36.09	29.32	
342		1	1.43	8.8	36.07	27.27	
346		1	0.47	7.9	37.52	29.62	
349		1	0.69	7.6	37.24	29.64	
351		1	2.32	7.6	37.14	29.54	
354		1	0.36	7.0	36.98	29.98	
357		1	0.44	7.0	36.92	29.92	
36		3	0.95	14.2	40.28	26.08	
360		1	0.48	6.7	36.88	30.18	
363		1	1.17	7.0	36.74	29.74	
364		1	0.00	7.0	36.38	29.38	
366		1	1.06	7.0	36.28	29.28	
369		1	0.09	8.2	36.54	28.34	
372		1	0.96	7.3	36.62	29.32	
375		1	0.88	7.6	36.49	28.89	
378		1	0.57	7.6	36.45	28.85	
379		1	0.86	6.3	36.54	30.22	
381		1	0.52	7.0	36.38	29.38	
384		1	0.91	7.6	36.24	28.64	
387		1	1.08	7.3	36.10	28.80	
39		3	0.74	14.0	40.31	26.31	
390		1	1.03	7.1	35.99	28.89	
393		1	0.83	7.6	36.02	28.42	
396		1	0.55	7.6	36.05	28.45	
399		1	1.02	8.5	36.06	27.56	
402		1	1.05	8.2	36.07	27.87	
405		1	1.17	8.0	35.92	27.92	
408		1	0.23	8.2	35.97	27.77	
410		1	0.00	8.2	36.01	27.81	
411		1	0.89	8.5	36.03	27.53	
414		1	1.18	9.0	35.94	26.94	
417		1	1.74	8.8	35.80	27.00	
42		3	0.79	14.0	40.32	26.32	
420		1	1.02	12.2	35.91	23.71	
423		1	0.79	8.8	35.88	27.08	
426		1	0.59	9.1	35.89	26.79	
428		1	0.04	12.2	35.87	23.67	
429		1	1.64	12.2	35.87	23.67	
43		3	0.00	14.0	40.30	26.30	
432		1	1.04	13.1	35.89	22.79	
435		1	1.01	10.0	35.87	25.87	
438		1	0.60	7.3	36.03	28.73	
439		1	0.53	8.5	36.10	27.60	
440		1	0.36	7.3	35.97	28.67	
441		1	0.91	8.0	35.89	27.92	

Node Name	Node Type	Area	Total Demand	Ground Level	Total Head	Available Head	Supply
444		1	1.32	10.0	35.81	25.81	
447		1	0.77	9.2	35.79	26.57	
45		3	2.12	16.4	40.32	23.91	
450		1	2.05	9.4	35.79	26.39	
453		1	2.13	8.2	35.97	27.77	
454		1	0.00	8.2	35.96	27.76	
456		1	1.16	8.5	35.88	27.38	
457		1	0.00	8.5	35.88	27.38	
459		1	0.00	8.5	35.66	27.16	
46		3	0.87	13.0	40.66	27.66	
460		1	1.03	7.2	35.66	28.47	
462		1	0.65	7.6	35.65	28.05	
465		11	15.83	9.6	39.85	30.26	
468		12	13.60	7.5	38.07	30.57	
47		3	0.65	13.0	41.04	28.04	
471		1	0.00	6.7	37.66	30.96	
472		1	0.00	6.1	37.49	31.35	
474		1	0.00	8.5	37.13	28.63	
477		10	234.64	9.2	36.62	27.40	
480		1	0.00	4.9	38.52	33.62	
483		1	0.00	5.5	37.24	31.74	
486		1	0.26	8.2	36.21	28.01	
487		1	0.00	7.6	36.21	28.65	
489		1	0.04	8.8	36.19	27.39	
49		3	0.25	14.0	40.58	26.58	
492		1	0.49	8.5	36.19	27.69	
495		1	0.26	8.2	36.19	27.99	
498		1	0.21	8.5	36.20	27.70	
5		1	0.00	32.6	43.16	10.56	
50		3	0.56	14.0	40.58	26.58	
501		1	0.00	8.2	36.20	27.95	
504		1	0.37	8.2	36.20	28.00	
507		1	0.13	8.2	36.20	28.00	
51		3	0.44	13.7	40.78	27.08	
510		1	0.65	8.2	36.21	28.01	
513		1	0.49	8.5	36.22	27.72	
516		1	0.39	8.5	36.21	27.71	
519		1	0.97	8.5	36.20	27.70	
522		1	0.00	8.8	36.65	27.85	
525		1	1.00	8.4	36.59	28.15	
528		1	0.09	8.6	36.49	27.89	
531		1	0.91	8.5	36.37	27.87	
532		1	0.17	8.8	36.29	27.49	
534		1	0.87	9.1	36.24	27.14	
535		1	0.26	8.2	36.22	28.02	
537		1	0.15	9.1	36.22	27.12	
54		3	0.79	13.7	40.76	27.06	
540		1	0.25	9.4	36.21	26.81	
543		1	0.00	8.5	36.20	27.70	
546		1	0.50	7.9	36.22	28.32	
549		1	1.41	8.8	36.21	27.41	
550		1	0.73	8.2	36.27	28.07	
552		1	0.59	9.4	36.24	26.84	

Node Name	Node Type	Area	Total Demand	Ground Level	Total Head	Available Head	Supply
555		1	0.30	9.4	36.22	26.82	
558		1	0.83	9.8	36.01	26.21	
561		1	0.90	9.7	35.95	26.25	
564		1	0.77	9.7	36.00	26.30	
565		1	0.00	9.8	36.04	26.24	
567		1	0.62	9.9	36.10	26.20	
568		1	0.63	9.8	36.09	26.29	
57		3	0.56	12.8	41.10	28.30	
570		1	0.75	9.4	36.07	26.67	
571		1	0.00	9.1	36.08	26.98	
572		1	1.03	9.1	36.08	26.98	
573		1	0.65	9.1	35.99	26.89	
576		1	0.52	9.4	36.00	26.60	
579		1	0.00	9.4	36.07	26.67	
585		1	0.85	9.8	35.92	26.12	
588		1	0.88	9.1	35.94	26.84	
591		1	0.68	8.8	36.00	27.20	
594		1	0.66	10.2	36.07	25.89	
597		1	0.63	9.1	36.08	26.98	
6		1	0.08	28.0	43.08	15.08	
600		1	1.12	8.5	36.09	27.59	
603		1	0.69	8.5	36.11	27.61	
606		1	0.25	11.3	35.92	24.62	
609		1	1.19	8.8	35.91	27.11	
612		1	0.46	11.3	35.89	24.59	
615		1	0.14	6.1	35.82	29.72	
618		1	1.21	6.7	35.82	29.12	
621		1	0.89	5.7	35.83	30.13	
624		1	0.61	8.2	35.84	27.64	
625		1	0.39	8.1	35.86	27.76	
627		1	0.12	7.6	35.88	28.28	
63		7	1.28	12.8	41.57	28.77	
630		1	0.55	7.0	35.90	28.90	
633		1	1.23	7.0	35.93	28.93	
636		1	1.68	7.0	36.03	29.03	
639		1	0.02	8.1	35.84	27.74	
642		1	0.07	8.1	35.88	27.78	
645		1	0.08	8.8	35.88	27.04	
648		1	0.16	9.1	35.89	26.79	
651		1	0.00	9.1	35.90	26.80	
654		1	0.53	9.1	35.92	26.82	
66		3	0.18	12.8	41.10	28.30	
69		3	0.17	14.0	42.05	28.05	
7		3	0.07	21.6	40.52	18.92	
72		3	0.31	14.6	42.26	27.62	
75		3	0.24	13.2	42.68	29.51	
78		6	2.91	14.3	42.19	27.89	
8		3	0.00	12.8	40.03	27.23	
80		2	0.14	14.0	39.69	25.69	
81		3	0.00	14.0	42.68	28.68	
84		2	0.38	18.6	39.81	21.21	
87		2	1.25	18.0	39.72	21.72	
9		3	2.30	12.8	40.00	27.20	

90	2	0.00	15.4	39.71	24.31
93	2	1.04	13.4	39.70	26.30
97	2	0.00	13.4	39.69	26.29
98	3	0.00	13.4	41.10	27.70
99	2	0.16	13.4	39.69	26.29
9A	9	0.47	12.8	40.00	27.20

NLYMSYS : TOWN C NETWORK

Snapshot time: 00/01:00C
Selected area: All
Special node selection: Off
Item type: Pipe

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
102	117A	171	364.	0.0350	5.23	0.23	0.13	0.4
102	154	243	377.	0.0500	-9.50	-0.20	-0.07	-0.2
102	99	219	102.	0.0410	0.80	0.02	0.00	0.0
114	117	97	183.	0.0210	0.29	0.04	0.01	0.0
117	117A	171	1.	0.0350	-3.35	-0.15	0.00	-0.2
117	120	114	278.	0.0990	2.06	0.20	0.14	0.5
117	123	171	670.	0.0350	1.02	0.04	0.01	0.0
117A	99	114	465.	0.0990	-1.48	-0.14	-0.13	-0.3
12	15	77	361.	3.5000	0.41	0.09	0.13	0.4
12	27	146	595.	0.0310	-2.41	-0.14	-0.12	-0.2
12	9A	77	139.	3.5000	1.18	0.25	0.41	3.0
123	126	171	336.	0.0350	0.80	0.03	0.00	0.0
126	127	114	726.	0.0990	0.67	0.07	0.05	0.1
127	128	114	490.	0.0990	0.55	0.05	0.03	0.1
128	177	114	1.	0.0990*	Pipe Unavailable *			
129	132	243	597.	0.5000	21.15	0.45	0.66	1.1
129	156	847	1060.	7.0000	402.39	0.71	1.16	1.1
129	2	847	154.	7.0000	-423.54	-0.75	-0.19	-1.2
132	135	243	680.	0.5000	20.45	0.44	0.71	1.0
135	138	105	209.	0.0210	1.64	0.19	0.10	0.5
135	141	243	178.	0.5000	18.45	0.40	0.15	0.9
138	144	105	122.	3.0000	1.05	0.12	0.05	0.4
141	144	155	153.	3.0000	0.45	0.02	0.00	0.0
141	147	243	331.	0.5000	17.92	0.39	0.27	0.8
144	145	150	1343.	3.0000	1.32	0.07	0.14	0.1
145	146	146	1020.	0.0310	0.55	0.03	0.02	0.0
15	21	102	274.	4.2000	-0.21	-0.03	-0.01	0.0
150	147	243	2035.	0.0500	-17.63	-0.38	-1.23	-0.6
150	153	243	452.	0.0500	12.46	0.27	0.15	0.3
150	84	165	535.	0.1210	3.65	0.17	0.13	0.2
153	114	97	461.	0.0210	1.32	0.18	0.23	0.5
153	154	243	104.	0.0500	10.61	0.23	0.03	0.2
156	159	362	920.	19.5	45.04	0.44	1.84	2.0
156	303	847	1260.	7.0000	357.35	0.63	1.09	0.9
159	162	362	1198.	1.2800	45.04	0.44	0.92	0.8
162	165	362	1190.	1.2800	45.02	0.44	0.92	0.8

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
165	166	362	273.	0.0500	30.97	0.30	0.07	0.2
165	233	295	570.	1.0000	12.73	0.19	0.10	0.2
166	168	285	927.	0.0500	30.97	0.48	0.72	0.8
168	171	285	765.	0.0500	30.45	0.48	0.58	0.8
171	174	285	640.	15.0	26.65	0.42	1.46	2.3
171	207	146	244.	0.0310	2.67	0.16	0.06	0.2
174	176	285	247.	8.0000	21.76	0.34	0.29	1.2
174	225	146	943.	35.0	1.86	0.11	0.72	0.8
176	218	310	15.	8.2000	5.12	0.07	0.00	0.0
177	180	114	403.	0.0990	0.00	0.00	0.00	0.0
180	183	114	450.	0.0990	-0.99	-0.10	-0.06	-0.1
183	186	114	184.	0.0990	-5.65	-0.55	-0.60	-3.3
186	189	146	268.	0.0310	-7.14	-0.42	-0.37	-1.4
189	192	219	442.	0.0410	-9.19	-0.24	-0.14	-0.3
192	198	219	418.	0.0410	-11.81	-0.31	-0.20	-0.5
192	213	146	61.	0.0310	0.94	0.06	0.00	0.0
198	176	295	458.	0.0490	-15.22	-0.22	-0.08	-0.2
198	215	219	59.	0.0410	2.80	0.07	0.00	0.0
1R	2	844	5.	7.0000	444.25	0.79	0.01	1.4
2	5	229	722.	7.0000	0.00	0.00	0.00	0.0
204	207	146	723.	0.0310	-2.06	-0.12	-0.11	-0.1
21	24	102	293.	4.2000	-1.24	-0.15	-0.24	-0.8
21	36	97	540.	0.0210	0.01	0.00	0.00	0.0
210	189	146	137.	0.0310	-1.03	-0.06	-0.01	0.0
210	210A	146	1.	5.5000	-0.05	0.00	0.00	0.0
210A	213	146	301.	5.5000	-2.41	-0.14	-0.14	-0.5
213	215	152	415.	5.5000	-2.76	-0.15	-0.20	-0.5
215	216	152	430.	5.5000	-1.30	-0.07	-0.05	-0.1
218	216	178	141.	5.8000	3.05	0.12	0.04	0.3
218	217	310	148.	8.2000	1.98	0.03	0.00	0.0
219	217	216	384.	15.0	-1.56	-0.04	-0.01	0.0
219	282	216	655.	15.0	-0.35	-0.01	0.00	0.0
222	225	146	429.	0.0310	-1.49	-0.09	-0.04	-0.1
226	225	146	295.	0.0310	-0.36	-0.02	0.00	0.0
226	228	146	266.	0.0310	-0.72	-0.04	-0.01	0.0
228	229	146	292.	0.0310	-0.93	-0.06	-0.01	0.0
228	231	146	381.	0.0310	-1.27	-0.08	-0.02	-0.1
229	231	146	185.	0.0310	-1.37	-0.08	-0.01	-0.1
231	279	146	218.	0.0310	-3.67	-0.22	-0.09	-0.4
233	234	216	1373.	1.0000	12.55	0.34	1.21	0.9
234	237	295	305.	25.0	18.70	0.27	0.37	1.2
234	240	216	412.	8.0000	11.09	0.30	0.57	1.4
234	252	114	264.	0.0500	4.24	0.42	0.48	1.8
237	243	295	284.	25.0	16.60	0.24	0.27	1.0
240	243	216	184.	8.0000	5.98	0.16	0.07	0.4
240	261	146	264.	0.0310	5.04	0.30	0.19	0.7
243	246	295	850.	15.0	12.62	0.19	0.37	0.4
243	270	102	342.	4.2000	2.14	0.26	0.81	2.4
246	249	127	312.	4.8000	1.86	0.15	0.18	0.6
246.	285	216	293.	8.0000	8.64	0.24	0.25	0.8
246	450	216	405.	6.0000	1.57	0.04	0.01	0.0
249	295	102	269.	4.2000	1.51	0.19	0.32	1.2
249	462	102	687.	4.2000	-0.28	-0.03	-0.03	0.0

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
252	255	114	159.	0.0500	3.70	0.36	0.23	1.4
255	258	146	643.	0.0310	0.89	0.05	0.02	0.0
258	259	146	210.	0.0310	0.00	0.00	0.00	0.0
261	263	146	323.	0.0310	2.91	0.17	0.09	0.3
263	255	114	240.	0.0500	-2.27	-0.22	-0.14	-0.6
263	264	114	214.	0.0500	5.18	0.51	0.56	2.6
264	267	114	131.	0.0500	3.96	0.39	0.21	1.6
267	276	219	340.	0.0410	-0.95	-0.03	0.00	0.0
267	279	114	460.	0.0500	3.14	0.31	0.49	1.1
27	24	139	66.	0.0310	1.69	0.11	0.01	0.1
27	49	219	224.	5.0000	-5.42	-0.14	-0.06	-0.3
27	50	114	224.	0.0500	-1.41	-0.14	-0.06	-0.3
27	8	102	378.	4.2000	1.58	0.19	0.49	1.3
270	264	102	428.	4.2000	-0.38	-0.05	-0.04	-0.1
270	271	102	233.	4.2000	1.03	0.13	0.13	0.6
271	273	97	90.	0.0210	1.03	0.14	0.03	0.3
273	276	97	467.	0.0210	0.26	0.04	0.01	0.0
276	289	219	372.	0.0410	-2.55	-0.07	-0.01	0.0
279	282	114	762.	0.0500	-1.78	-0.17	-0.29	-0.4
282	294	216	70.	8.0000	-2.76	-0.08	-0.01	-0.1
285	284	146	188.	0.0310	0.18	0.01	0.00	0.0
288	285	216	265.	8.0000	-8.02	-0.22	-0.19	-0.7
289	288	216	262.	8.0000	-7.34	-0.20	-0.16	-0.6
291	289	216	39.	8.0000	-4.79	-0.13	-0.01	-0.3
291	300	89	112.	0.0180	-0.62	-0.10	-0.02	-0.2
293	291	216	412.	8.0000	-3.95	-0.11	-0.07	-0.2
294	293	216	696.	8.0000	-3.84	-0.11	-0.12	-0.2
296	295	125	236.	0.0260	-1.15	-0.09	-0.03	-0.1
296	297	97	39.	0.0210	1.15	0.16	0.02	0.4
297	298	89	224.	0.0180	0.62	0.10	0.04	0.2
300	298	102	73.	0.0210	-0.62	-0.08	-0.01	-0.1
303	304	374	20.	450.0	37.27	0.34	0.33	16.4
303	465	847	1240.	7.0000	320.08	0.57	0.86	0.7
304	306	374	2382.	5.0000	37.27	0.34	1.59	0.7
306	309	374	800.	5.0000	35.18	0.32	0.48	0.6
306	480	139	1442.	0.0310	2.09	0.14	0.28	0.2
309	311	374	1023.	5.0000	37.25	0.34	0.68	0.7
309	480	139	1024.	0.0310	-2.09	-0.14	-0.20	-0.2
311	312	295	1.	0.0500*	Pipe Unavailable *			
311	346	295	126.	0.0500	37.25	0.55	0.12	0.9
312	315	374	327.	8.0000	32.05	0.29	0.19	0.6
312	483	363	1131.	0.0540	-42.35	-0.41	-0.49	-0.4
312	487	146	1.	6.0000*	Pipe Unavailable *			
315	312	241	327.	8.0000	-9.75	-0.21	-0.19	-0.6
315	321	241	456.	8.0000	8.39	0.18	0.20	0.4
315	550	146	337.	0.0310	5.60	0.33	0.30	0.9
318	315	374	194.	8.0000	-27.06	-0.25	-0.08	-0.4
318	324	374	474.	8.0000	27.86	0.25	0.21	0.4
321	324	241	266.	8.0000	7.50	0.16	0.09	0.3
324	327	241	192.	8.0000	7.82	0.17	0.07	0.4
324	333	374	490.	8.0000	23.25	0.21	0.15	0.3
324	387	139	300.	0.0310	3.89	0.26	0.17	0.6
327	330	241	137.	3.0000	8.26	0.18	0.04	0.3

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
330	333	241	158.	3.0000	7.45	0.16	0.04	0.2
333	336	241	187.	3.0000	5.00	0.11	0.02	0.1
333	423	89	282.	0.0180	1.44	0.23	0.25	0.9
333	439	374	283.	0.1720	16.47	0.15	0.02	0.1
336	339	241	326.	0.1460	3.85	0.08	0.01	0.0
336	609	102	436.	4.2000	0.94	0.12	0.19	0.4
339	342	219	144.	0.0410	5.54	0.15	0.02	0.1
339	439	374	226.	0.1720	-15.94	-0.14	-0.01	-0.1
339	636	203	108.	6.4000	6.28	0.19	0.06	0.6
342	453	146	336.	0.0310	3.12	0.19	0.10	0.3
342	654	102	283.	4.2000	0.99	0.12	0.15	0.5
346	349	294	305.	0.0500	36.77	0.54	0.28	0.9
349	351	295	179.	0.0500	28.19	0.41	0.10	0.6
351	354	295	320.	0.0500	25.87	0.38	0.15	0.5
354	357	295	127.	0.0300	25.51	0.37	0.06	0.5
357	360	295	121.	0.0500	22.11	0.32	0.04	0.4
36	43	77	318.	3.5000	-0.17	-0.04	-0.02	-0.1
360	234	295	196.	0.0500	21.63	0.32	0.07	0.3
363	357	97	88.	0.0210	-2.96	-0.40	-0.18	-2.1
364	363	97	421.	0.0210	-1.79	-0.25	-0.36	-0.9
364	366	97	119.	0.0210	1.79	0.25	0.10	0.9
366	384	97	229.	0.0210	0.74	0.10	0.04	0.2
372	349	146	375.	0.0310	-7.89	-0.47	-0.61	-1.6
372	369	146	288.	0.0310	3.01	0.18	0.08	0.3
372	379	146	178.	0.0310	3.91	0.23	0.08	0.5
375	318	146	146.	0.0310	0.99	0.06	0.01	0.0
375	369	146	175.	0.0310	-2.93	-0.17	-0.05	-0.3
378	375	97	128.	0.0210	-1.06	-0.15	-0.04	-0.3
378	379	97	162.	0.0210	-1.43	-0.20	-0.09	-0.6
378	387	97	361.	0.0210	1.92	0.26	0.35	1.0
379	381	97	232.	0.0210	1.61	0.22	0.16	0.7
381	384	97	388.	0.0210	1.10	0.15	0.14	0.4
384	237	97	534.	0.0210	-1.17	-0.16	-0.20	-0.4
384	396	97	171.	0.0210	2.09	0.29	0.19	1.1
387	390	139	329.	0.1160	2.76	0.18	0.11	0.3
39	36	97	148.	0.0210	0.78	0.11	0.03	0.2
39	42	175	206.	5.8000	-1.52	-0.06	-0.02	-0.1
390	408	216	146.	0.1350	6.17	0.17	0.03	0.2
393	390	216	337.	0.1350	4.44	0.12	0.03	0.1
393	396	216	197.	0.1350	-5.27	-0.14	-0.03	-0.1
396	399	216	89.	0.1350	-3.72	-0.10	-0.01	-0.1
399	402	216	69.	0.1350	-6.08	-0.17	-0.01	-0.2
399	411	152	234.	5.4000	1.33	0.07	0.03	0.1
402	243	216	447.	0.1350	-7.12	-0.20	-0.10	-0.2
405	387	102	89.	4.2000	-1.97	-0.24	-0.18	-2.0
408	410	102	351.	4.2000	-0.44	-0.05	-0.04	-0.1
408	414	216	158.	0.1350	6.38	0.17	0.03	0.2
411	410	97	311.	0.0210	0.44	0.06	0.02	0.1
414	420	216	258.	0.1350	5.20	0.14	0.03	0.1
417	405	102	369.	4.2000	-0.80	-0.10	-0.13	-0.3
417	423	102	168.	4.2000	-0.95	-0.12	-0.08	-0.5
42	45	77	212.	4.2000	0.09	0.02	0.01	0.0
420	426	216	135.	0.1350	4.19	0.11	0.01	0.1

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
423	426	102	332.	4.2000	-0.29	-0.04	-0.02	0.0
426	432	216	108.	0.1350	3.31	0.09	0.01	0.1
428	429	139	65.	0.1160	-0.04	0.00	0.00	0.0
429	432	139	491.	0.1160	-0.74	-0.05	-0.02	0.0
432	435	139	369.	0.1160	1.01	0.07	0.02	0.1
432	441	216	298.	0.1350	0.52	0.01	0.00	0.0
438	339	216	118.	6.0000	-7.00	-0.19	-0.06	-0.5
438	429	102	337.	4.2000	0.94	0.12	0.16	0.5
438	440	216	213.	6.0000	5.46	0.15	0.06	0.3
440	441	216	283.	6.0000	5.23	0.14	0.08	0.3
440	453	146	165.	0.0310	-0.13	-0.01	0.00	0.0
441	444	216	393.	6.0000	4.54	0.12	0.08	0.2
441	456	102	231.	4.2000	0.30	0.04	0.01	0.1
444	447	216	380.	6.0000	2.04	0.06	0.02	0.0
444	460	97	360.	0.0210	1.18	0.16	0.15	0.4
447	450	216	231.	6.0000	0.48	0.01	0.00	0.0
447	462	102	420.	4.2000	0.78	0.10	0.14	0.3
45	43	77	246.	3.5000	0.17	0.04	0.02	0.1
45	46	97	278.	0.0310	-2.19	-0.30	-0.34	-1.2
454	453	146	100.	0.0020	-0.86	-0.05	0.00	0.0
454	456	102	224.	4.2000	0.86	0.11	0.09	0.4
457	456	77	251.	3.5000	0.00	0.00	0.00	0.0
459	460	102	108.	4.2000	0.00	0.00	0.00	0.0
46	47	97	544.	0.0200	-1.59	-0.22	-0.38	-0.7
460	462	102	389.	4.2000	0.15	0.02	0.01	0.0
465	468	847	2835.	7.0000	304.25	0.54	1.78	0.6
468	471	847	725.	7.0000	290.65	0.52	0.42	0.6
471	472	363	403.	0.0540	42.35	0.41	0.17	0.4
471	474	847	965.	15.0	248.30	0.44	0.53	0.5
471	480	139	883.	0.0310*	Pipe Unavailable *			
472	483	363	564.	0.0540	42.35	0.41	0.24	0.4
474	477	847	638.	50.0	234.64	0.42	0.52	0.8
486	487	146	145.	6.0000	0.00	0.00	0.00	0.0
49	42	97	180.	0.0210	2.40	0.33	0.26	1.4
49	50	114	1.	0.0500	-0.29	-0.03	0.00	0.0
49	54	219	340.	5.0000	-7.79	-0.21	-0.18	-0.5
492	489	146	268.	0.0310	0.04	0.00	0.00	0.0
495	492	146	220.	0.0310	0.53	0.03	0.00	0.0
50	51	114	340.	0.0500	-2.26	-0.22	-0.20	-0.6
501	498	146	100.	0.0310	0.21	0.01	0.00	0.0
501	519	146	281.	0.0310	0.31	0.02	0.00	0.0
504	501	146	124.	0.0310	0.52	0.03	0.00	0.0
504	507	146	182.	0.0310	0.13	0.01	0.00	0.0
51	57	114	390.	0.0500	-2.70	-0.26	-0.32	-0.8
510	504	146	133.	0.0310	1.02	0.06	0.01	0.0
513	510	146	111.	0.0310	1.67	0.10	0.01	0.1
513	516	146	272.	6.0000	0.65	0.04	0.01	0.0
513	528	146	415.	6.0000	-2.81	-0.17	-0.27	-0.7
516	486	146	448.	6.0000	0.26	0.02	0.00	0.0
516	519	146	129.	0.0310	1.45	0.09	0.01	0.1
519	495	146	158.	0.0310	0.79	0.05	0.00	0.0
522	474	216	660.	0.1350	-13.66	-0.37	-0.49	-0.7
522	525	216	80.	0.1350	13.66	0.37	0.06	0.7

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
525	528	216	150.	0.1350	12.66	0.35	0.10	0.6
531	528	216	170.	3.0000	-9.75	-0.27	-0.13	-0.7
531	532	216	180.	3.0000	7.32	0.20	0.08	0.4
531	558	102	737.	0.0210	1.53	0.19	0.36	0.5
532	534	244	237.	3.0000	7.15	0.15	0.05	0.2
534	535	216	79.	3.0000	5.68	0.16	0.02	0.3
534	564	77	305.	3.5000	0.60	0.13	0.24	0.8
535	537	216	31.	3.0000	4.68	0.13	0.01	0.2
535	549	146	290.	0.0310	0.74	0.04	0.01	0.0
537	540	229	103.	6.9000	2.91	0.07	0.01	0.1
537	568	102	94.	4.2000	1.62	0.20	0.13	1.4
54	46	97	165.	0.0200	1.47	0.20	0.10	0.6
54	57	219	390.	5.0000	-10.05	-0.27	-0.34	-0.9
540	567	152	279.	5.4000	2.50	0.14	0.11	0.4
543	327	219	418.	3.0000	0.56	0.01	0.00	0.0
543	540	219	206.	3.0000	-1.50	-0.04	0.00	0.0
543	572	102	270.	4.2000	0.94	0.12	0.13	0.5
546	516	146	79.	0.0310	1.46	0.09	0.01	0.1
549	546	146	137.	0.0310	-0.67	-0.04	0.00	0.0
550	546	146	234.	0.0310	2.64	0.16	0.05	0.2
550	552	146	198.	0.0310	2.23	0.13	0.03	0.2
552	555	146	181.	0.0310	1.65	0.10	0.02	0.1
555	540	146	134.	0.0310	1.35	0.08	0.01	0.1
561	558	97	374.	0.0210	-0.70	-0.10	-0.06	-0.2
561	585	97	412.	0.0210	0.42	0.06	0.03	0.1
564	561	102	264.	4.2000	0.63	0.08	0.06	0.2
564	565	102	94.	4.2000	-0.81	-0.10	-0.03	-0.3
565	573	102	280.	4.2000	0.58	0.07	0.05	0.2
567	568	102	110.	4.2000	0.40	0.05	0.01	0.1
567	570	152	190.	5.4000	1.49	0.08	0.03	0.1
568	565	102	49.	4.2000	1.39	0.17	0.05	1.0
57	47	146	360.	0.0310	2.24	0.13	0.06	0.2
57	63	219	222.	5.0000	-15.73	-0.42	-0.47	-2.1
57	66	114	226.	0.0500	0.18	0.02	0.00	0.0
570	579	152	46.	5.4000	1.06	0.06	0.00	0.1
571	570	97	226.	0.0210	0.32	0.04	0.01	0.0
571	597	146	310.	0.0310	-0.19	-0.01	0.00	0.0
572	571	102	88.	4.2000	0.13	0.02	0.00	0.0
572	600	102	322.	4.2000	-0.22	-0.03	-0.01	0.0
573	576	97	100.	0.0210	-0.63	-0.09	-0.01	-0.1
573	588	102	297.	4.2000	0.56	0.07	0.05	0.2
576	579	97	115.	0.0210	-1.43	-0.20	-0.07	-0.6
576	591	146	275.	0.0310	0.29	0.02	0.00	0.0
579	594	152	291.	5.4000	-0.38	-0.02	0.00	0.0
585	588	102	216.	4.2000	-0.43	-0.05	-0.02	-0.1
588	591	102	196.	4.2000	-0.74	-0.09	-0.06	-0.3
591	594	102	104.	4.2000	-1.14	-0.14	-0.07	-0.7
594	606	102	160.	4.2000	1.36	0.17	0.15	1.0
597	594	254	190.	7.2000	3.52	0.07	0.01	0.1
597	600	241	182.	0.1440	-4.35	-0.10	-0.01	-0.1
6	2	374	760.	0.0500	-20.72	-0.19	-0.08	-0.1
6	75	374	3938.	0.0500	20.64	0.19	0.40	0.1
603	333	241	128.	0.1440	-6.38	-0.14	-0.01	-0.1

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
603	600	241	256.	0.1440	5.69	0.12	0.02	0.1
606	609	102	330.	4.2000	0.25	0.03	0.00	0.0
606	612	102	62.	4.2000	0.85	0.11	0.02	0.4
612	627	102	199.	4.2000	0.39	0.05	0.01	0.1
615	618	190	216.	0.1320	-0.14	0.00	0.00	0.0
618	621	190	222.	0.1320	-1.34	-0.05	0.00	0.0
621	624	203	259.	0.4160	-2.23	-0.07	-0.01	0.0
624	625	203	204.	6.4000	-2.86	-0.09	-0.02	-0.1
627	625	203	192.	0.4160	3.25	0.10	0.02	0.1
627	630	203	160.	6.4000	-2.82	-0.09	-0.02	-0.1
627	645	102	129.	4.2000	-0.16	-0.02	0.00	0.0
63	69	219	196.	5.0000	-17.01	-0.45	-0.48	-2.5
630	633	203	205.	6.4000	-3.37	-0.10	-0.03	-0.2
633	636	203	327.	6.4000	-4.60	-0.14	-0.10	-0.3
639	624	96	202.	0.0210	-0.02	0.00	0.00	0.0
642	645	97	125.	0.0210	-0.07	-0.01	0.00	0.0
645	648	97	241.	0.0210	-0.30	-0.04	-0.01	0.0
648	651	102	130.	4.2000	-0.46	-0.06	-0.02	-0.1
651	654	102	172.	4.2000	-0.46	-0.06	-0.02	-0.1
66	98	146	552.	0.0310	0.00	0.00	0.00	0.0
69	72	219	83.	5.0000	-17.18	-0.46	-0.21	-2.5
7	27	139	928.	0.1160	-0.07	0.00	0.00	0.0
72	75	219	119.	5.0000	-20.40	-0.54	-0.42	-3.5
72	78	146	254.	0.0310	2.91	0.17	0.07	0.3
75	81	219	5.	5.0000	0.00	0.00	0.00	0.0
8	9	146	226.	3.0000	1.58	0.09	0.04	0.2
80	81	219	1.	5.0000*	Pipe Unavailable *			
80	99	219	214.	0.0410	-0.14	0.00	0.00	0.0
84	87	165	440.	0.1210	3.27	0.15	0.09	0.2
87	90	165	172.	0.1210	2.03	0.09	0.01	0.1
9	9A	146	1.	3.0000	-0.71	-0.04	0.00	0.0
90	93	165	110.	0.1210	2.03	0.09	0.01	0.1
93	99	165	111.	0.1210	0.98	0.05	0.00	0.0
97	98	146	1.	0.0310*	Pipe Unavailable *			
97	99	146	61.	0.0310	0.00	0.00	0.00	0.0

NLYMSYS : TOWN C NETWORK

Snapshot time: 00/01:00C
Selected Area: All
Special node selection: Off
Item type: Valve

Inlet Node	Outlet Node	Valve Type	Setting	Inlet Head	Outlet Head	Flow	Status	Control Node

NLYMSYS : TOWN C NETWORK

Snapshot time: 00/01:00C
Selected Area: All
Special node selection: Off
Item type: Var Resvr

Node Name	Bottom Water Level	Top Water Level	Actual Water Level
*1R	38.10	45.00	43.1625

A.2. NETWORKS FOR THE EXAMPLES IN CHAPTER 5

A.2.1. Test networks used in Case 1

The schematics for these networks are shown in the main text. The main characteristics are as follows.

Test network 1

- 16 nodes: constant elevation 100m AOD; demand flow of 30 l/s at all nodes except 1, 7, 10 and RES.
- 34 links: lenght 500 m, C factor 100, diameter 300 mm for all.
- 1 reservoir: elevation 130 m AOD, bottom water level 130 m AOD, top water level 180 m AOD, reservoir level at 170 m AOD, unlimited volume.
- Daily demand profile, starting at 0:00h and ending at 24:00h: 0.6, 0.6, 0.6, 0.6, 0.6, 0.6, 0.8, 1.1, 1.4, 1.5, 1.4, 1.3, 1.2, 1.1, 1.1, 1.0, 1.0, 1.2, 1.4, 1.2, 1.0, 0.9, 0.8, 0.7, 0.6.

Test network 2

- 49 nodes: constant elevation 100m AOD; demand flow of 30 l/s at all nodes except 1, 5, 6, 7, 10, 12, 13, 14, 16, 18, 20, 21, 23, 26, 27, 29, 30, 31, 34, 38, 39, 40, 42, 44, 47 and RES.
- 106 links: lenght 500 m, C factor 100, diameter 300 mm for all except 800mm 1-RES; 600mm 1-2, 1-8, 1-9; 500mm 2-10, 9-10, 8-15, 8-16, 9-16, 2-3, 2-9, 8-9.
- 1 reservoir: elevation 130 m AOD, bottom water level 130 m AOD, top water level 180 m AOD, reservoir level at 170 m AOD, unlimited volume.
- Daily demand profile, starting at 0:00h and ending at 24:00h: 0.6, 0.6, 0.6, 0.6, 0.6, 0.6, 0.8, 1.1, 1.4, 1.5, 1.4, 1.3, 1.2, 1.1, 1.1, 1.0, 1.0, 1.2, 1.4, 1.2, 1.0, 0.9, 0.8, 0.7, 0.6.

Test network 3

- 70 nodes: constant elevation 100m AOD; demand flow of 30 l/s at all nodes except 1, 5, 7, 10, 12, 13, 14, 16, 18, 20, 21, 23, 26, 27, 29, 30, 31, 34, 38, 39, 40, 42, 44, 47, 53, 56, 57, 60, 65, 68 and RES.
- 154 links: lenght 500 m, C factor 100, diameter 300 mm for all except 800mm 1-RES; 600mm 1-2, 1-8, 1-9; 500mm 2-10, 9-10, 8-15, 8-16, 9-16, 2-3, 2-9, 8-9.
- 1 reservoir: elevation 130 m AOD, bottom water level 130 m AOD, top water level 180 m AOD, reservoir level at 170 m AOD, unlimited volume.
- Daily demand profile, starting at 0:00h and ending at 24:00h: 0.6, 0.6, 0.6, 0.6, 0.6, 0.6, 0.8, 1.1, 1.4, 1.5, 1.4, 1.3, 1.2, 1.1, 1.1, 1.0, 1.0, 1.2, 1.4, 1.2, 1.0, 0.9, 0.8, 0.7, 0.6.

Test network 4

As network 3, plus one extra link and one extra reservoir (RES2) with same characteristics as RES1 except water level at 190m AOD. New link RES2-64 has length 500m, C of 100 and diameter 800mm, all other diameters 300mm except RES1-1 800mm, 1-2, 1-8, 1-9 500mm.

A.2.2. East Edinburgh network used in Case 2

The tables below include the topological data, network characteristics and the hydraulic solution for the average demand load.

```
Snapshot time: 00/01:00C *Warnings exist for this snapshot time*
Selected area: All
Special node selection: Off
Item type: Node
```

Node Name	Node Type	Area	Total Demand	Ground Level	Total Head	Available Head	Supply
100	RESR	1	0.00	121.6	121.60	0.00	874.15
101		1	0.00	106.1	120.99	14.92	
102		1	0.00	78.3	119.96	41.63	
103		1	0.00	64.9	119.04	54.12	
104		1	0.00	105.8	121.19	15.42	
105		1	0.00	56.4	120.58	64.19	
106		1	12.79	75.0	119.72	44.74	
107		1	13.65	64.0	117.00	52.99	
108		1	10.47	63.4	117.30	53.90	
109		1	0.00	70.1	119.61	49.51	
109A		1	0.00	70.1	119.61	49.51	
110		1	0.00	54.0	121.11	67.16	
110A		1	0.00	54.0	119.81	65.86	
110B		1	0.00	56.7	118.80	62.11	
111		1	10.36	45.5	117.00	71.45	
112		1	0.00	48.5	116.99	68.53	
113		1	29.07	72.2	119.18	46.94	
114		1	0.00	80.8	117.39	36.62	
115		1	11.79	85.9	116.95	31.00	
116		1	34.21	68.0	117.49	49.52	
116B		1	0.00	59.7	118.92	59.22	
117		1	0.00	76.5	117.26	40.76	
118		1	0.00	73.5	117.08	43.62	
118A		1	0.00	73.5	117.04	43.54	
119		1	0.00	77.1	117.14	40.03	
120		1	0.00	74.7	116.07	41.37	
121		1	0.00	72.9	115.31	42.41	
122		1	0.00	72.5	116.47	43.93	
123		1	0.00	76.2	117.11	40.91	
124		1	32.20	45.7	113.93	68.23	
125		1	0.00	32.9	113.93	81.03	
126		1	23.69	81.7	116.73	35.03	
127		1	0.00	79.9	116.58	36.68	
128		1	13.17	36.6	113.71	77.11	
129		1	9.96	89.0	116.46	27.46	
130		1	0.00	78.0	115.79	37.79	
131		1	0.00	66.8	114.23	47.43	
132		1	9.86	58.5	113.62	55.12	
133		1	10.16	59.1	112.70	53.57	
134		1	0.00	61.3	112.57	51.31	

Node Name	Node Type	Area	Total Demand	Ground Level	Total Head	Available Head	Supply
135		1	0.00	52.7	112.65	59.95	
136	PRVU	1	0.00	30.8	109.62	78.82	
136A	PRVD	1	0.00	30.8	64.60	33.80	
137		1	0.00	20.1	62.89	42.79	
138		1	75.33	13.4	55.70	42.30	
139		1	0.00	15.9	62.33	46.43	
140		1	29.68	44.2	112.72	68.52	
141		1	0.00	27.7	112.72	85.02	
142		1	0.00	12.5	62.37	49.87	
143		1	0.00	8.8	62.25	53.45	
144		1	0.00	24.1	62.85	38.75	
145		1	37.01	7.9	62.14	54.24	
146		1	0.00	27.1	63.03	35.91	
147		1	0.00	20.4	62.91	42.51	
148		1	0.00	10.1	62.96	52.86	
149		1	0.00	36.6	63.59	26.99	
150		1	31.12	23.5	62.98	39.48	
151		1	0.00	10.9	62.99	52.09	
152		1	0.00	11.3	62.99	51.69	
153		1	0.00	5.2	63.15	57.95	
154		1	7.66	35.9	64.11	28.21	
155		1	19.66	21.3	64.50	43.20	
156		1	3.68	5.5	63.22	57.72	
157		1	44.59	3.9	63.16	59.26	
158		1	14.28	31.1	65.56	34.46	
159		1	25.79	19.8	63.73	43.93	
160		1	0.00	4.3	64.07	59.77	
161		1	20.24	61.0	112.49	51.53	
162U		1	0.00	54.8	112.71	57.91	
163	PRVU	1	0.00	35.4	112.81	77.41	
163A	PRVD	1	0.00	35.4	65.88	30.48	
164		1	24.87	0.0	62.05	62.05	
165		1	19.12	28.7	70.82	42.12	
166		1	0.00	44.5	112.99	68.49	
167		1	0.00	32.9	113.09	80.19	
168	PRVU	1	0.00	30.8	112.73	81.93	
168A	PRVD	1	0.00	30.8	64.60	33.80	
169		1	4.06	29.6	73.19	43.59	
170		1	1.64	45.1	116.54	71.44	
171		1	1.65	47.9	114.77	66.87	
172		1	0.00	33.2	113.49	80.29	
173	PRVU	1	0.00	32.6	113.46	80.86	
173A	PRVD	1	0.00	32.6	66.40	33.80	
174		1	29.50	9.4	65.43	55.98	
175	PRVU	1	0.00	31.4	113.65	82.25	
175A	PRVD	1	0.00	31.4	73.39	41.99	
176		1	0.00	47.2	116.36	69.16	
177		1	24.19	46.9	115.80	68.90	
178		1	7.77	39.9	115.84	75.94	
179		1	5.97	41.8	116.22	74.46	
180		1	0.00	43.6	116.39	72.79	
181		1	2.77	37.2	116.10	78.90	
182		1	13.16	37.2	115.58	78.38	

Node Name	Node Type	Area	Total Demand	Ground Level	Total Head	Available Head	Supply
183	PRVU	1	0.00	43.6	115.75	72.15	
183A	PRVD	1	0.00	43.6	77.40	33.80	
184		1	46.64	36.0	68.24	32.27	
185		1	20.23	57.9	113.18	55.28	
186		1	0.00	55.8	113.99	58.19	
187		1	19.22	43.9	112.32	68.42	
188		1	0.00	71.9	114.52	42.62	
189		1	7.97	56.9	113.84	56.94	
190		1	43.96	64.0	114.00	50.00	
191		1	0.00	56.1	113.88	57.78	
191A		1	0.00	59.4	113.88	54.44	
192		1	0.00	43.3	112.69	69.39	
193		1	0.00	39.3	112.69	73.39	
194	PRVU	1	0.00	42.6	112.59	69.99	
194A	PRVD	1	0.00	42.6	76.40	33.80	
195		1	31.39	22.6	74.37	51.81	
196	THVD	1	10.00	59.4	116.40	56.96	
196A	THVU	1	10.00	59.4	115.57	56.13	
197		1	0.00	76.2	116.46	40.26	
198		1	0.00	63.4	116.43	53.03	
199		1	0.00	61.3	114.00	52.74	
200		1	6.62	63.4	114.98	51.58	
201		1	6.50	63.3	114.78	51.48	
202		1	6.50	63.2	114.78	51.58	
CT		1	0.00	92.1	116.45	24.39	
DUNSAP		1	0.00	86.3	112.71	26.45	

EI5N1SYS:

Snapshot time: 00/01:00C *Warnings exist for this snapshot time*
Selected area: All
Special node selection: Off
Item type: Pipe

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
100	101	762	810.	110.0	310.74	0.68	0.61	0.8
100	104	559	480.	110.0	147.48	0.60	0.41	0.9
100	104	610	480.	115.0	194.03	0.66	0.41	0.9
100	109	381	1840.	100.0	55.22	0.48	1.99	1.1
100	109A	559	1840.	110.0	166.68	0.68	1.99	1.1
101	102	610	630.	110.0	262.12	0.90	1.03	1.6
101	104	610	620.	110.0	-108.90	-0.37	-0.20	-0.3
101	105	689	1170.	110.0	157.52	0.42	0.41	0.4
102	103	610	670.	115.0	250.42	0.86	0.92	1.4
102	106	229	360.	105.0	11.70	0.28	0.24	0.7
103	107	305	390.	20.0	14.40	0.20	2.03	5.2
103	108	610	1400.	115.0	236.02	0.81	1.73	1.2
104	110	559	1410.	110.0	33.55	0.14	0.08	0.1
104	110A	610	1410.	110.0	199.06	0.68	1.38	1.0
105	180	610	4090.	85.0	157.52	0.54	4.19	1.0
106	110A	152	840.	80.0	-1.09	-0.06	-0.08	-0.1

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
107	111	305	950.	105.0	0.75	0.01	0.00	0.0
108	112	610	280.	115.0	225.55	0.77	0.32	1.1
109	109A	381	10.	65.0	9.91	0.09	0.00	0.1
109	110	610	330.	1.0000	-4.14	-0.01	-1.49	-4.5
109	116B	381	790.	100.0	49.45	0.43	0.69	0.9
109A	113	559	300.	100.0	176.59	0.72	0.43	1.4
110	111	508	700.	10.0	29.41	0.15	4.11	5.9
110A	110B	610	870.	100.0	197.97	0.68	1.01	1.2
110B	114	610	1000.	90.0	197.97	0.68	1.41	1.4
111	112	254	1450.	80.0	1.31	0.03	0.02	0.0
111	112	229	1450.	80.0	0.99	0.02	0.02	0.0
111	115	508	1500.	90.0	17.50	0.09	0.06	0.0
112	170	508	224.	120.0	197.70	0.98	0.45	2.0
112	176	254	550.	90.0	17.66	0.35	0.63	1.1
112	180	229	1730.	80.0	6.25	0.15	0.60	0.3
112	180	229	1730.	80.0	6.25	0.15	0.60	0.3
113	116	559	1360.	90.0	147.52	0.60	1.70	1.2
114	115	381	250.	70.0	50.81	0.45	0.45	1.8
114	119	610	220.	75.0	147.16	0.50	0.25	1.1
115	120	508	450.	35.0	56.52	0.28	0.87	1.9
116	117	559	240.	80.0	113.31	0.46	0.23	1.0
116B	197	381	2300.	90.0	49.45	0.43	2.46	1.1
117	118	559	170.	75.0	113.31	0.46	0.18	1.1
118	121	381	1230.	70.0	45.06	0.40	1.76	1.4
118	126	508	460.	70.0	68.24	0.34	0.35	0.8
118A	119	610	120.	55.0	-92.85	-0.32	-0.10	-0.9
118A	122	610	660.	55.0	92.85	0.32	0.57	0.9
119	123	610	180.	70.0	54.31	0.19	0.04	0.2
120	123	381	190.	20.0	-26.44	-0.23	-1.03	-5.4
120	124	457	1505.	80.0	82.96	0.51	2.15	1.4
121	122	508	610.	55.0	-87.87	-0.43	-1.16	-1.9
121	188	381	490.	70.0	48.19	0.42	0.79	1.6
121	188	508	700.	70.0	84.74	0.42	0.79	1.1
122	129	508	600.	55.0	4.99	0.02	0.01	0.0
123	127	406	340.	35.0	27.87	0.22	0.53	1.6
124	125	229	340.	60.0	0.00	0.00	0.00	0.0
124	128	457	190.	60.0	55.05	0.34	0.22	1.1
124	131	229	890.	55.0	-4.29	-0.10	-0.31	-0.3
126	127	381	140.	35.0	19.25	0.17	0.15	1.1
126	129	406	630.	65.0	25.30	0.20	0.26	0.4
127	130	406	190.	35.0	47.12	0.36	0.78	4.1
128	140	406	810.	60.0	41.88	0.32	0.99	1.2
129	198	406	400.	65.0	10.01	0.08	0.03	0.1
129	CT	406	230.	65.0	10.32	0.08	0.02	0.1
130	131	305	360.	55.0	35.87	0.49	1.56	4.3
130	132	229	890.	50.0	11.26	0.27	2.18	2.4
131	132	305	180.	55.0	31.58	0.43	0.62	3.4
132	135	305	260.	55.0	32.98	0.45	0.97	3.7
133	190	229	640.	50.0	-10.16	-0.25	-1.30	-2.0
134	135	305	340.	30.0	-3.92	-0.05	-0.08	-0.2
134	136	305	750.	55.0	34.08	0.47	2.96	3.9
134	196A	305	310.	30.0	-30.15	-0.41	-2.99	-9.7
135	136	305	540.	55.0	41.25	0.56	3.03	5.6

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
135	140	305	240.	80.0	-12.20	-0.17	-0.07	-0.3
136A	137	305	100.	55.0	75.33	1.03	1.71	17.1
137	138	305	420.	55.0	75.33	1.03	7.19	17.1
139	142	305	480.	95.0	-7.65	-0.10	-0.04	-0.1
139	143	305	910.	95.0	7.65	0.10	0.08	0.1
140	141	229	440.	50.0	0.00	0.00	0.00	0.0
142	144	305	720.	95.0	-22.48	-0.31	-0.48	-0.7
142	145	305	750.	95.0	14.83	0.20	0.23	0.3
143	145	229	210.	80.0	7.65	0.19	0.11	0.5
144	146	305	510.	95.0	-15.94	-0.22	-0.18	-0.4
144	147	229	160.	80.0	-6.54	-0.16	-0.06	-0.4
145	148	229	500.	80.0	-14.53	-0.35	-0.82	-1.6
146	149	406	540.	70.0	-44.75	-0.35	-0.56	-1.0
146	150	406	110.	70.0	28.81	0.22	0.05	0.5
147	148	229	870.	80.0	-2.46	-0.06	-0.05	-0.1
147	150	229	440.	80.0	-4.08	-0.10	-0.07	-0.2
148	151	406	260.	95.0	-16.99	-0.13	-0.03	-0.1
149	154	406	500.	70.0	-44.75	-0.35	-0.52	-1.0
150	152	406	640.	95.0	-6.39	-0.05	-0.01	0.0
151	152	406	100.	95.0	-1.28	-0.01	0.00	0.0
151	153	305	480.	95.0	-15.71	-0.22	-0.16	-0.3
152	156	229	460.	80.0	-7.67	-0.19	-0.23	-0.5
153	156	305	220.	95.0	-14.81	-0.20	-0.07	-0.3
153	157	305	720.	95.0	-0.91	-0.01	0.00	0.0
154	158	406	1040.	70.0	-52.41	-0.40	-1.45	-1.4
155	158	305	580.	130.0	-53.11	-0.73	-1.06	-1.8
155	159	305	990.	130.0	33.45	0.46	0.77	0.8
156	157	229	430.	80.0	4.04	0.10	0.07	0.2
156	159	305	680.	120.0	-30.19	-0.41	-0.51	-0.7
157	160	305	680.	120.0	-41.46	-0.57	-0.91	-1.3
158	163A	508	440.	125.0	-119.80	-0.59	-0.32	-0.7
159	160	305	720.	115.0	-22.54	-0.31	-0.34	-0.5
160	165	305	2090.	115.0	-63.99	-0.88	-6.76	-3.2
161	DUNSAP	305	460.	100.0	-20.24	-0.28	-0.23	-0.5
162U	166	305	550.	100.0	-20.24	-0.28	-0.27	-0.5
162U	DUNSAP	500	1.	100.0	20.24	0.10	0.00	0.0
163	166	508	240.	125.0	-119.80	-0.59	-0.18	-0.7
164	168A	153	100.	90.0	-24.87	-1.35	-2.55	-25.5
165	169	305	180.	70.0	-83.11	-1.14	-2.37	-13.1
166	167	508	100.	125.0	-140.04	-0.69	-0.10	-1.0
167	168	229	100.	90.0	24.87	0.60	0.36	3.6
167	172	508	300.	125.0	-164.91	-0.81	-0.40	-1.3
169	175A	457	100.	70.0	-87.17	-0.53	-0.20	-2.0
170	171	508	965.	125.0	196.06	0.97	1.77	1.8
171	172	508	710.	125.0	194.41	0.96	1.28	1.8
172	173	305	20.	90.0	29.50	0.40	0.02	1.2
173A	174	305	800.	90.0	29.50	0.40	0.97	1.2
175	180	457	3150.	110.0	-87.17	-0.53	-2.73	-0.9
176	177	254	440.	85.0	17.66	0.35	0.56	1.3
177	178	254	200.	85.0	-6.53	-0.13	-0.04	-0.2
178	179	254	450.	85.0	-14.30	-0.28	-0.39	-0.9
179	180	254	100.	85.0	-20.27	-0.40	-0.16	-1.6
180	181	204	2120.	80.0	2.77	0.08	0.29	0.1

From node	To Node	Diam.	Length	Frict.	Flow	Velocity	Headloss	Headloss Gradient
180	182	229	2120.	80.0	6.58	0.16	0.81	0.4
180	182	229	2120.	80.0	6.58	0.16	0.81	0.4
180	183	229	50.	85.0	46.64	1.13	0.64	12.7
183A	184	229	720.	85.0	46.64	1.13	9.16	12.7
185	186	305	540.	55.0	-20.23	-0.28	-0.81	-1.5
186	187	305	1220.	55.0	19.22	0.26	1.67	1.4
186	188	508	790.	65.0	-58.94	-0.29	-0.53	-0.7
186	189	457	930.	60.0	19.49	0.12	0.16	0.2
188	190	381	570.	55.0	27.68	0.24	0.52	0.9
188	190	356	570.	55.0	23.15	0.23	0.52	0.9
188	190	356	570.	55.0	23.15	0.23	0.52	0.9
189	190	229	440.	50.0	-4.05	-0.10	-0.16	-0.4
189	191	457	380.	60.0	-15.82	-0.10	-0.04	-0.1
189	192	204	560.	50.0	7.53	0.23	1.14	2.0
189	192	305	560.	55.0	23.86	0.33	1.14	2.0
190	191	381	370.	55.0	15.82	0.14	0.12	0.3
190	199	381	370.	55.0	0.00	0.00	0.00	0.0
191	191A	457	720.	60.0	0.00	0.00	0.00	0.0
192	193	204	870.	50.0	0.00	0.00	0.00	0.0
192	194	305	30.	55.0	31.39	0.43	0.10	3.4
194A	195	305	600.	55.0	31.39	0.43	2.03	3.4
196	198	381	350.	70.0	-10.01	-0.09	-0.03	-0.1
196A	197	381	1230.	90.0	-40.15	-0.35	-0.89	-0.7
197	CT	381	200.	70.0	9.30	0.08	0.02	0.1
200	201	229	804.	100.0	6.50	0.16	0.20	0.2
200	202	229	804.	100.0	6.50	0.16	0.20	0.2
200	CT	229	400.	70.0	-19.62	-0.48	-1.47	-3.7

EI5N1SYS:

Snapshot time: 00/01:00C *Warnings exist for this snapshot time*
Selected Area: All
Special node selection: Off
Item type: Valve

Inlet Node	Outlet Node	Valve Type	Setting	Inlet Head	Outlet Head	Flow	Status	Control Node
175	175A	PRV	73.39	113.65	73.39	87.17	Actv	
163	163A	PRV	65.88	112.81	65.88	119.80	Actv	
173	173A	PRV	66.40	113.46	66.40	29.50	Actv	
183	183A	PRV	77.40	115.75	77.40	46.64	Actv	
136	136A	PRV	64.60	109.62	64.60	75.33	Actv	
194	194A	PRV	76.40	112.59	76.40	31.39	Actv	
168	168A	PRV	64.60	112.73	64.60	24.87	Actv	
196A	196	THV	0.10	115.57	116.40	-0.01	Actv	

A.2.3. SCRWWA network used in Case 3

[NODES]

ID	Elev. ft.	Demand gpm	Demand Pattern
1	50	-694.4	2
2	100	8	
3	60	14	
4	60	8	
5	100	8	
6	125	5	
7	160	4	
8	110	9	
9	180	14	
10	130	5	
11	185	34.78	
12	210	16	
13	210	2	
14	200	2	
15	190	2	
16	150	20	
17	180	20	
18	100	20	
19	150	5	
20	170	19	
21	150	16	
22	200	10	
23	230	8	
24	190	11	
25	230	6	
27	130	8	
28	110	0	
29	110	7	
30	130	3	
31	190	17	
32	110	17	
33	180	1.5	
34	190	1.5	
35	110	0	
36	110	1	

[TANKS]

ID	Elev. ft.	Init. Level	Min. Level	Max. Level	Diam. ft.
26	235	56.7	50	70	50

[QUALITY]

First Node	Last Node	Fluoride mg/L
1	36	1.0

[SOURCES]

ID	Fluoride mg/L	Source Pattern
1	1.0	3

[PIPES]

ID	Head Node	Tail Node	Length ft.	Diam. in.	Rough. Coeff.
1	1	2	2400	12	100
2	2	5	800	12	100
3	2	3	1300	8	100
4	3	4	1200	8	100
5	4	5	1000	12	100
6	5	6	1200	12	100
7	6	7	2700	12	100
8	7	8	1200	12	140
9	7	9	400	12	100
10	8	10	1000	8	140
11	9	11	700	12	100
12	11	12	1900	12	100
13	12	13	600	12	100
14	13	14	400	12	100
15	14	15	300	12	100
16	13	16	1500	8	100
17	15	17	1500	8	100
18	16	17	600	8	100
19	17	18	700	12	100
20	18	32	350	12	100
21	16	19	1400	8	100
22	14	20	1100	12	100
23	20	21	1300	8	100
24	21	22	1300	8	100
25	20	22	1300	8	100

26	24	23	600	12	100
27	15	24	250	12	100
28	23	25	300	12	100
29	25	26	200	12	100
30	25	31	600	12	100
31	31	27	400	8	100
32	27	29	400	8	100
34	29	28	700	8	100
35	22	33	1000	8	100
36	33	34	400	8	100
37	32	19	500	8	100
38	29	35	500	8	100
39	35	30	1000	8	100
40	28	35	700	8	100
41	28	36	300	8	100

[PATTERNS]

Demand pattern	
1	1.26 1.04 .97 .97 .89 1.19 1.28
1	.67 .67 1.34 2.46 .97 .92 .68
1	1.43 .61 .31 .78 .37 .67 1.26 1.56
1	1.19 1.26 .6 1.1 1.03 .73
1	.88 1.06 .99 1.72 1.12 1.34 1.12
1	.97 1.04 1.15 .91 .61 .68 .46
1	.51 .74 1.12 1.34 1.26 .97 .82
1	1.37 1.03 .81 .88 .81 .81

Pump flow pattern	
2	.96 .96 .96 .96 .96 .96 .62 0 0 0 0 0 .8 1 1
2	1 1 .15 0 0 0 0 0 0 .55 .92 .92 .92 .92 .9
2	.9 .45 0 0 0 0 0 .7 1 1 1 1 .2 0 0
2	0 0 0 0 .74 .92 .92 .92 .92 .92

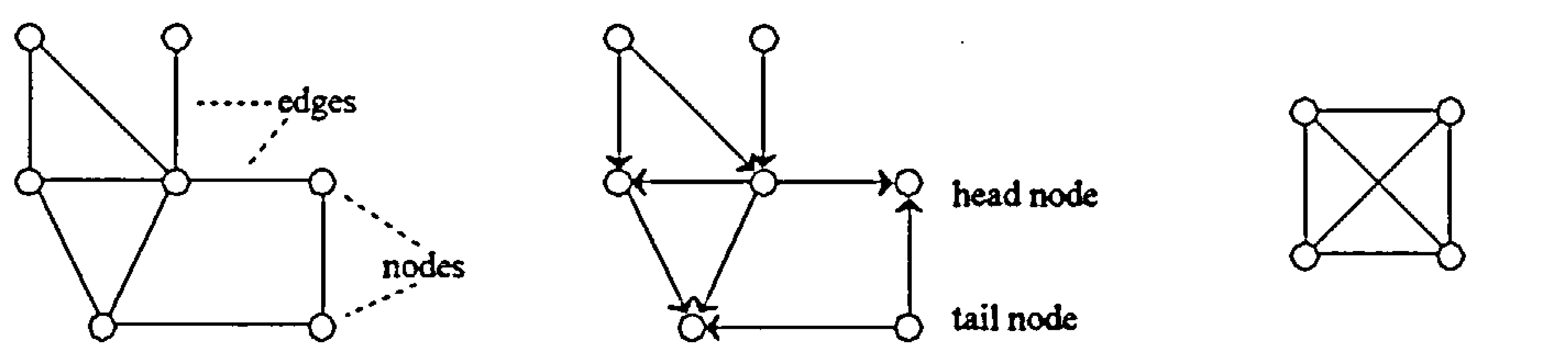
Fluoride source pattern	
3	.98 1.02 1.05 .99 .64 .46 .35 .35
3	.35 .35 .35 .35 .17 .17 .13 .13 .13 .15
3	.15 .15 .15 .15 .15 .15 .15 .12 .1 .08
3	.11 .09 .09 .08 .08 .08 .08 .08 .08
3	.09 .07 .07 .09 .09 .09 .09 .09 .09
3	.09 .09 .09 .08 .35 .72 .82 .92 1

APPENDIX B

A SUMMARY OF GRAPH THEORY TERMINOLOGY AND CONCEPTS

This text presents a short reference of the most used terminology and concepts in graph theory, with the purpose of clarifying some of the methods referred to in the text. Henley and Williams (1973) may be consulted for a complete and structured introduction to graph theory and its applications in engineering, including reliability analysis and optimisation, while Billington and Allan (1983) provide an overview of reliability evaluation using that as well as other techniques.

A ***graph*** *G(N,E)* consists of a set of ***nodes*** *N* and a set of ***edges*** *E*, an edge being a link between any pair of nodes of *N* (Fig.B.1.a). A graph is ***directed*** (Fig.B.1.b) if the pair of nodes corresponding to any edge is ordered, in which case one of the nodes is the ***head***



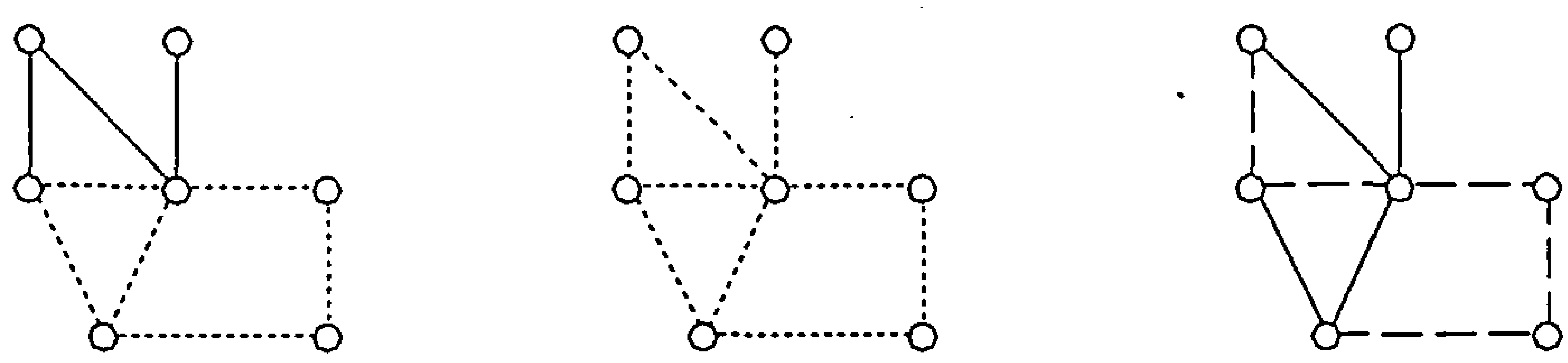
a) Graph b) Directed graph c) Regular in degree

Fig.B.1

(destination) and the other is the ***tail*** (origin). If the graph is not directed it is termed ***undirected***. Many properties are common to both directed and undirected graphs.

The ***degree*** of a node is the number of incident edges on that node. A graph is said to be ***regular*** in degree if each node has the same degree (Fig.B.1.c).

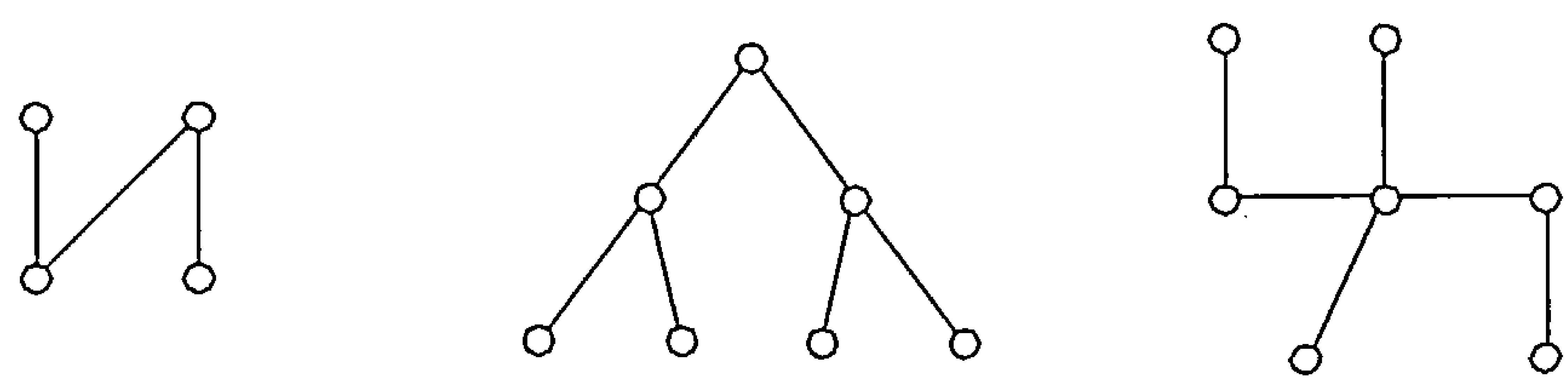
A ***subgraph*** $\hat{G}$ of graph G (Fig.B.2.a) consists of nodes and edges that belong to, or form subsets of, N and E. If the node sets of $\hat{G}$ and G are the same, $\hat{G}$ is a ***spanning subgraph*** of G (Fig.B.2.b). On the other hand, a graph is said ***complete*** if its set of edges contains all possible edges that can be formed between the nodes of its node set. The graph previously shown in Fig.B.1.c is complete.



a) Subgraph (hatched)　　b) Spanning subgraph　　c) Path (hatched line)

Fig.B.2

A ***path*** from node n_1 to node n_k is a sequence of edges such that $\{(n_1, n_2), (n_2, n_3), (n_3, n_4), ..., (n_{k-1}, n_k)\}$. A path may be directed or undirected, just as a graph. Paths are ***edge-disjoint*** or ***node-disjoint*** if they consist of distinct edges or nodes, respectively. The path shown in Fig.B.2.c is both edge- and node-disjoint. A ***circuit*** is a path where first and last node coincide, $n_1 = n_k$, and a ***simple circuit*** consists of all distinct nodes. A graph is ***acyclic*** if it contains no circuits (Fig.B.3.a).



a) Acyclic graph　　b) Tree　　c) Spanning tree

Fig.B.3

An undirected graph is said to be *connected* if there is at least one path between any two nodes. Examples of connected graphs have been previously shown in Figs.B.1.a and B.1.c. A ***component*** is the maximal connected subgraph in which a particular node is included. A *disconnected* graph contains more than one component.

A ***tree*** (Fig.B.3.b) is an undirected, connected acyclic graph. The edges of a tree are called ***branches***. A ***spanning tree*** of a graph is a spanning subgraph which is also a tree (Fig.B.3.c shows a spanning tree of the graph introduced in Figs.B.1.a and B.1.b). An ***arborescence*** is an acyclic directed graph in which the ***root*** is the only node with no entering edge and all other nodes have exactly one entering edge. A ***spanning arborescence*** (or ***directed spanning tree***) is a spanning subgraph which is also an arborescence.

A ***cut-set*** is a set of edges whose removal from a connected graph leaves it disconnected, provided that no other subset of these edges disconnects the graph. A cut-set can be identified between any two mutually exclusive subsets of nodes by means of the minimum number of edges whose removal disables all paths between those two subsets. There are usually considerably more cut-sets than nodes in a graph, the upper boundary being the number of different node partitions. One cut-set of particular interest is the one by which a particular node is isolated from all others. The number of edges in that cut-set equals the number of incident edges on the node, or that node's degree.

A network is a graph in which all the edges possess some properties in addition to their two end nodes. In case of water distribution networks, edges represent the hydraulic elements such as pipelines, pumps and other devices, linking any two points (tipically intersections, changes in diameter, tanks, device boundaries, etc.) which are conventionally designated nodes. The loops of a hydraulic network are no more than circuits, and a looped network constitutes a connected graph where each edge belongs to a circuit.

A *minimal cut-set* is a cut-set that would not disconnect the graph if at least one of its components were to be restored. All the components of a minimal cut-set must fail for it to disconnect the graph.

The probability of failure of the generic minimal cut-set C_i is:

$$p(C_i) = \prod_{n=1}^{N_i} PF_n \tag{B.1}$$

where PF_n is the probability of failure of the n^{th} component in the i^{th} minimal cut-set, which has N_i components.

The reliability of a system (network, etc.) represented by a graph in the terms described above can be calculated. Given the definition of minimal cut-set, the failure of any combination of the possible minimal cut-sets will cause the system to fail. For a system with NC minimal cut-sets, the reliability R is given by:

$$R = 1 - p\left[\bigcup_{i=1}^{NC} C_i\right] \tag{B.2}$$

with the elementary probabilities given by Eq.B.1. The above expression can be calculated exactly for any NC, but the number of terms involved escalates quickly. Approximations can be successfully used when the individual component reliabilities are high, which means reduced probabilities of failure of the minimal cut-sets and smaller significance of the above products. A lower bound on reliability may be found through the following approximation, obtained discarding the second and higher order products in Eq.E.2:

$$R = 1 - \sum_{i=1}^{NC} p(C_i) \tag{B.3}$$

APPENDIX C

CALCULATION OF MAXIMUM ENTROPY FLOWS IN NETWORKS

This text presents the method introduced by Tanyimboh (1993) for calculating maximum entropy flows in single source networks. In fact, its applicability to multiple-source networks depends solely on what is expected from the definitions of maximum entropy flows and their relationship with the concepts of redundancy and reliability, as discussed in the main text, and the method may be modified to account for uncertainty in the source supplies, as will be seen shortly. The method is only briefly explained here, and the interested reader may refer to Tanyimboh (1993) or Tanyimboh and Templeman (1993a).

The original procedure for single source networks is a 3-step procedure as described next.

i) The nodes in the network are sequentially numbered by flow precedence starting at the source. The algorithm to be followed consists of numbering the source node with 1, increasing the index number by one and attributing it to the next node whose upstream nodes have all been numbered, and repeating this step until the last node is reached.

ii) Next, the nodes thus numbered are weighed in such a way that the individual weights represent the number of distinct paths to that node, NP_n. This is done as follows:

1 - Start off with the source node, whose weight is 1 (NP_n=1)

2 - Move on to the next node in the sequence, and calculate NP_n as the sum of the number of paths leading to each of the upstream contributing nodes:

$$NP_n = \sum_{j \in U^n} NP_j \tag{C.1}$$

3 - Repeat until the last node is reached.

iii) Finally, the flow is evenly distributed among all the paths feeding the nodes, starting the process at the last node in the numbered sequence:

1 - Set n to the number of nodes.

2 - Calculate the nodal flow Q_n:

$$Q_n = \sum_{k \in D^n} q_{nk} \tag{C.2}$$

3 - Calculate q_{jn} by distributing the total flow Q_n reaching the node via the NP_n paths among the contributing nodes, weighted by their respective number of paths NP_j:

$$q_{jn} = Q_n \frac{NP_j}{NP_n} \tag{C.3}$$

4 - Set n to n-1 and return to 2 until the first node is reached.

To extend this algorithm to multiple source networks with maximum entropy source supply flows, the network is modified to include a new node, a super-source (as introduced in Chapter 5 of the main text). In the new, modified network, the algorithms above are applied exactly as described but taking into consideration the new node as if it were real. The maximum entropy flows thus produced from the super-source to the real source nodes are the maximum entropy source supply flows of the real network. That is to say, they correspond to the maximum uniformity of supply paths to the demand nodes in the network. It must be noted this does not imply that all the source supply flows should be equal.

On the other hand, it is important to notice the similarity between this procedure and the path-based formulation for the flow-joining processes, which measures the uncertainty generated by the diversity of paths supplying a given node. Equation C.2 above yields nothing else than the flow distribution that maximises its direct counterpart equation 6.30, by applying the principle that entropy is maximised by the uniformity of probabilities.